U.S. DEPARTMENT OF COMMERCE
National Technical Information Service

PB-286 560

Management of Small Waste Flows

Wisconsin Univ -Madison

Prepared for

Municipal Environmental Research Lab, Cincinnati, OH

Sep 78

United States
Environmental Protection
Agency

Municipal Environmental Research
Laboratory
Cincinnati OH 45268

EPA-600/2-78-173
September 1978

PB 286 560

Research and Development

EPA

Management of Small Waste Flows

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TECHNICAL REPORT DATA
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1. REPORT NO. EPA-600/2-78-173	2.	3. RECIPIENT'S ACCESSION NO. PB286560
4. TITLE AND SUBTITLE MANAGEMENT OF SMALL WASTE FLOWS		5. REPORT DATE September 1978 (Issuing Date) 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Small Scale Waste Management Project		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS University of Wisconsin - Madison University of Wisconsin - Extension Madison, Wisconsin 53706		10. PROGRAM ELEMENT NO. C611B 11. CONTRACT/GRANT NO. R802874
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Enginronmental Research Laboratory--Cin, OH Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268		13. TYPE OF REPORT AND PERIOD COVERED Final (7/71 - 6/77) 14. SPONSORING AGENCY CODE EPA/600/14

15. SUPPLEMENTARY NOTES

Project Officer: James Kreissl (513) 684-7614

16. ABSTRACT

This report is a compilation of laboratory and field investigations conducted at the University of Wisconsin since 1971. As its primary objective, the research program was to conceive, evaluate and develop satisfactory methods for the on-site treatment and disposal of wastewaters, regardless of the site constraints. The studies were subdivided into several categories including characterization of household and commercial wastewaters, assessment of wastewater treatment alternatives, evaluation of soils for treatment and disposal of wastewater, estimation of infiltrative capacities of soils, design and operation of alternative systems dependent upon soil design and operation of alternative systems not dependent upon soil, management of on-site disposal systems and institutional and regulatory control of on-site systems.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS		b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Sewage	Law (Juris prudence)	On-site sewage disposal	13B, 06I,
Water analysis	Sanitation	wastewater characterization	48E, 06M
Sewage disposal		alternative treatment systems	89B
Sewage treatment		subsurface soil systems	
Soil science		surface discharge	
Microbiology		community-wide management	

18. DISTRIBUTION STATEMENT Release to Public	19. SECURITY CLASS *(This Report)* Unclassified	21. NO. OF PAGES 810
	20. SECURITY CLASS *(This page)* Unclassified	22. PRICE PCA99

☆ U. S. GOVERNMENT PRINTING OFFICE: 1978-757-140/1423 Region No. 5-II

EPA-600/2-78-173
September 1978

MANAGEMENT OF SMALL WASTE FLOWS

by

Small Scale Waste Management Project
University of Wisconsin-Madison
University of Wisconsin-Extension
Madison, Wisconsin 53706

Grant No. R-802874

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FOREWORD

The U.S. Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

The report represents a comprehensive study of individual household wastewater management which will have a significant impact on the quality of life in rural area of the United States. It will increase the existing body of knowledge relating to the understanding of conventional and alternative on-site wastewater systems and reduce the wasteful spending of resources to superimpose expensive and technically inappropriate urban solutions on rural areas.

Francis T. Mayo
Director
Municipal Environmental Research
Laboratory

ABSTRACT

This report is a compilation of laboratory and field investigations conducted at the University of Wisconsin since 1971. As its primary objective, the research program was to conceive, evaluate and develop satisfactory methods for the on-site treatment and disposal of wastewaters, regardless of the site constraints. The studies were subdivided into several categories including characterization of household and commercial wastewaters, assessment of wastewater treatment alternatives, evaluation of soils for treatment and disposal of wastewater, estimation of infiltrative capacities of soils, design and operation of alternative systems dependent upon soil design and operation of alternative systems not dependent upon soil, management of on-site disposal systems and institutional and regulatory control of on-site systems.

Wastewater characterization was performed at eleven rural homes in Wisconsin. Flow data was collected for a total of 434 days; wastewater quality was monitored over a total of 35 days at four of these homes. Characterization of selected public establishments was based primarily upon a review of literature supplemented with some field data generation. A methodology for estimating commercial waste characteristics was delineated.

On-site wastewater treatment systems were evaluated: (a) in a controlled system employing a wastewater simulator, and (b) at ten field sites located throughout the State of Wisconsin. Eleven septic tanks, eleven aerobic units, a chemical unit and four sand filters were studied.

The soil was evaluated as a treatment and disposal medium for wastewater. Field and laboratory studies were made to determine the ability of different soils to remove bacteria, viruses and the nutrients, nitrogen and phosphorus under different loadings and soil temperatures. Infiltration rates through biologically clogged soil surfaces were determined and methods to reduce the severity of clogging were evaluated. Field procedures for the measurement of soil hydraulic conductivity were developed and evaluated. The results of these studies were used to improve the design, installation and operation of conventional and alternative soil absorption systems.

It was recognized early in the study that appropriate institutional controls are a prerequisite for effective on-site waste management. Based upon nation-wide surveys and a review of current regulatory options, a number of institutional control strategies are outlined and discussed. Land use implications of the new technology developed is also evaluated.

This report was submitted in fulfillment of Grant No. R-802874 by the University of Wisconsin under the partial sponsorship of the U.S. Environmental Protection Agency. This report covers a period from July, 1971 to June 1977 and work is continuing.

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LIST OF ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

B.D.	-- bulk density
BOD_5	-- 5-day biochemical oxygen demand
BOD_5 F	-- filtered BOD_5
BOD_5 U	-- unfiltered BOD_5
BV	-- bed volumes
cc	-- cubic centimeter
C.I.	-- confidence interval
cap, c	-- capita
cm	-- centimeter
COD	-- chemical oxygen demand
Coli	-- coliform organisms
CPM	-- counts per minute
cu ft	-- cubic feet
CV	-- coefficient of variation
d	-- day
D	-- width, dispersion coefficient
D.O.	-- dissolved oxygen
ES	-- effective size
FC	-- fecal coliform
FS	-- fecal streptococcus
ft	-- feet
ft^2	-- square feet
FU	-- fixture unit
g	-- gram, gravitational constant
G	-- geometric mean
gal	-- gallon
gpd	-- gallons per day
gpm	-- gallons per minute
h, hr	-- hour
H	-- hydraulic head
Ho	-- positive hydraulic head
ΔH	-- change in hydraulic head
hp	-- horsepower
ID	-- inside diameter
in	-- inch
JTU	-- Jackson Turbidity Unit
Kcal	-- kilocalorie
Kg	-- kilogram
Krad	-- kilorad
KW	-- kilowatt
KWhr	-- kilowatt-hour
L	-- liter, distance
lb	-- pound
Log	-- Logrithm
L/min	-- liters per minute
m	-- meter
m^2	-- square meter
m^3	-- cubic meter
mbar	-- millibar
mm	-- millimeter
MBAS	-- Methylene Blue Active Substance
mg	-- milligram
mg/L	-- milligrams per liter
min	-- minute
mL	-- milliliter
MLSS	-- mixed liquor suspended solids
mM	-- milli moles
MPN	-- most probable number
M_s	-- oven dry weight of soil
mv	-- millivolt
M_w	-- mass of water
N	-- nitrogen, normal concentration, number
nm	-- nanometer
OD	-- outside diameter
ORP, Eh	-- Oxidation-Reduction Potential
P	-- phosphorus
p	-- pressure
PBS	-- phosphate-buffered saline

ABBREVIATIONS (CONTINUED)

PFU	-- plaque forming unit
pH	-- negative log of the hydrogen ion activity
PMK	-- primary monkey kidney
Po-1	-- polivirus 1
Po-2	-- polivirus 2
Ps. a.	-- Pseudomonas aeruginosa
PT	-- ponding time
r	-- radius
s	-- submergence potential
S.a.	-- Staphyloccoccus aureus
SD, S_L	-- standard deviation
STE	-- septic tank effluent
SVI	-- sludge volume index
t	-- time
TBC	-- total bacterial count
TC	-- total coliforms
temp	-- temperature
TOC	-- total organic carbon
TOC U	-- unfiltered TOC
TOC F	-- filtered TOC
TS	-- total solids
TSS	-- total suspended solids
TVS	-- total volatile solids
TVSS	-- total volatile suspended solids
UC	-- uniformity coefficient
V_a	-- volume of soil air
V_s	-- volume of solids
V_w	-- volume of water
W	-- watt

SYMBOLS

^{o}C	-- degrees celsius (centigrade)
δ	-- surface tension of water
$\nabla\phi$	-- hydraulic gradient
ε	-- water-filled porosity
δ	-- specific weight
K	-- hydraulic conductivity
K_b	-- hydraulic conductivity of a crust or barrier
K_p	-- hydraulic conductivity after ponding occurs
K_s	-- hydraulic conductivity of a soil under a crust or barrier

SYMBOLS (CONTINUED)

K_{sat}	-- saturated hydraulic conductivity
M	-- molar concentration
μ	-- micro, micron
μL	-- microliter
μM	-- micromoles
μm	-- micrometer
η	-- viscosity
#	-- number
%	-- percentage
ψ_c	-- critical moisture potential
ψ_g	-- gravitational potential
ψ_m	-- matric or capillary potential
ψ_Ω	-- overburden potential
ψ_o	-- osmotic potential
ψ_p	-- pressure potential
ψ_t	-- total potential of water
Q	-- flow rate
q_b	-- flux through crust or barrier
q_s	-- flux through soil
$\vec{q}$	-- vector specific discharge
R_b	-- hydraulic resistance of barrier or crust
ρ_w	-- density of water
θ	-- volumetric moisture content
θ_v	-- percent soil moisture by volume
θ_w	-- percent soil moisture by weight
V	-- flux
v'	-- average velocity
Z	-- height above datum

ACKNOWLEDGMENTS

In 1971, the State of Wisconsin provided research funds to the University of Wisconsin to commence investigations into the on-site disposal of wastewater, at which time the Small Scale Waste Management Project was established. Since that time, substantial funding has also been provided by the Wisconsin Department of Natural Resources, the Upper Great Lakes Regional Commission, and the United States Environmental Protection Agency. This report is based upon research efforts conducted under these funding agencies. The authors wish to acknowledge the generous support of these agencies and the research efforts of the Small Scale Waste Management staff--

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SECTION 1

INTRODUCTION

When rural electrification brought a clean power source to the farm, families were able to install pressurized water systems in their homes for the first time. The use of modern indoor plumbing became commonplace, but with no sewerage available the generated wastewater created a disposal problem. To provide disposal, cesspools were commonly used, but subsurface irrigation systems were occasionally installed to increase the soil absorption area following the cesspool or buried tank. This was the forerunner of the modern septic tank - soil absorption system.

The treatment and disposal systems were usually constructed by the homeowners themselves or by local entrepreneurs in accordance with plans furnished by federal and state departments of health. A septic tank was installed to protect the soil absorption field by acting as a settling basin. Following the tank, trenches were dug wide enough to accomodate drain tile which was laid directly on the exposed trench bottom in open joint fashion. The joints were covered with tar paper before backfilling. No aggregate was used. A drain tile length of 40 feet per person was considered sufficient despite the different soil conditions encountered, though some health departments suggested that in "dense" soils the trench be excavated somewhat deeper and wider and the bottom filled with coarse aggregate before laying the tile. The purpose of the aggregate was to provide a porous media through which the septic tank effluent could flow to increase the infiltration area and to provide storage of the liquid until it could seep away.

Not surprisingly, failures, characterized by surfacing effluent were quite common. These concerned Henry Ryon of the New York Health Department because of the potential public health hazards they created. He felt better design criteria could be developed relating soil type to the amount of absorption area required. To develop these criteria, Ryon devised the percolation test and correlated the soils percolation rate measured by the test with its ability to accept septic tank effluent (Federick, 1948). From these correlations he made recommendations for absorption area required per person versus the measured percolation rate.

Ryon's method using the percolation test for sizing of the absorption field improved performance of the septic tank system and was quickly adopted. throughout much of the United States. Failures still occurred, but because the system served isolated residences, they went largely unnoticed. However, after World War II, there was an unprecedented expansion of small lot housing developments in the metropolitan fringes beyond the reach of sewers. Incidences of widespread failures were reported and became a concern because of the health hazards and nuisances they created. The number of failures indicated that on-site disposal systems were poorly understood. Responding to the need for improved practices, the federal government funded extensive studies of septic tank systems by the Public Health Service (Weibel, et al., 1949; Bendixen, et al., 1950; Weibel, et al., 1954) and the University of California (Winneberger, et al., 1960; McGauhey and Winneberger, 1965). The results of the studies culminated in the writing of the Manual of Septic Tank Practice first published by the U.S. Public Health Service in 1957 and revised in 1967 in which recommendations for improved practices were made. Recommendations included a standardized percolation test procedure, consideration of other site characteristics, decreased maximum loading rates, absorption field sizing based on the size and type of the building to be served rather than its present use, and that systems not be permitted in soils with percolation rates slower than 60 minutes per inch. These recommendations were adopted by most state health departments.

By 1970, approximately 16.6 million housing units or 25 percent of all housing units in the United States disposed of their wastewaters via septic tank-soil absorption systems (Cooper and Rezek, 1977). The use of these systems is growing at a rate of about one-half million new systems per year (Patterson, et al., 1971). This rate is ever increasing due to an emerging trend of population movement to rural areas (Beale and Fuguitt, 1975).

If used in conjunction with good construction procedures and good maintenance programs, the recommendations made in the Manual of Septic Tank Practice (U.S. Public Health Service, 1967), prove adequate for design of septic tank systems in most cases. However, it is estimated that only about 32 percent of the total land area of the United States has soils meeting the criteria recommended by the Manual. Consequently, with no alternative system, the septic tank system is often used where failures can be expected to occur because of the pressure to develop new lands. In addition, cases of contaminated wells attributed to inadequately treated septic tank effluent and nutrient enrichment of lakes from near shore developments has prompted a shift in emphasis from merely absorption of the effluent to providing treatment as well. Clearly, there is a need to understand better how the conventional septic tank-soil absorption system functions, why it fails to absorb or adequately treat the wastewater, what alternative systems might be employed in "unsuitable" soil areas, and how these systems might be more adequately regulated if the public health and environment is to be protected in rural areas.

This report is a compilation of studies which have addressed these questions. Although comprehensive in its scope, the report does not represent a complete coverage of all aspects of the problem. A number of the investigations that were initiated before or during the period of this grant have been completed and are presented within the text; others are in progress and preliminary results have been outlined. It would be presumptious to assume that this report is an end product, but rather it is an intermediate step in seeking solutions for the treatment and disposal of small wastewater flows.

SECTION 2

CONCLUSIONS

The Small Scale Waste Management Project, initiated in 1971 through State of Wisconsin funding, has undertaken extensive laboratory and field investigations of the treatment and disposal of small wastewater flows. Because of the comprehensive nature of these investigations and the diversity of projects that have been undertaken, the details of the work have been placed in the appendices to this report. The body of this report presents a distillation of these studies with appropriate conclusions. To list separately more abbreviated conclusions from this comprehensive study, would be counter productive because the statements would be out of context and misleading.

The investigation of the treatment and disposal of small wastewater flows is an on-going program at the University of Wisconsin. As such, new problem areas are being defined, experiments are being designed and evaluations are underway. Although a number of technical and institutional solutions have been found, it would be far too presumptious to assume that this report is an end product. More realistically, it is the beginning of an awareness of problems and possibilities which lie ahead in the evaluation of alternatives for treatment and disposal of small wastewater flows.

SECTION 3

RECOMMENDATIONS

This study represents an increasing concern for the problems with providing satisfactory treatment and disposal of small wastewater flows. The results show the potential of only a few alternative technologies for meeting the goal of low cost solutions. Much work remains to be done.

There is a need for additional demonstration of non-water carriage toilets, very low flow toilets and recycle systems including installation and operational characteristics, operation and maintenance costs, characterization of waste residues, and use acceptance. Alternatives for the treatment of grey water needs to be evaluated and demonstrated in field installations. The treatment and/or disposal of residues from biological toilets, chemical toilets, recycle toilets and black waters from very low flow toilet systems should be studied in both field and laboratory experiments.

Among the viable unit processes technologically available, on-site field demonstration of such processes as ozonation and iodination (disinfection), phosphorus sorption and ion exchange would be desirable.

Though soil absorption fields have been used for treatment and disposal of small wastewater flows since the late 1800's, several areas of needed research remain. These include: simplified tests for measurement of unsaturated hydraulic conductivity of soil, measurement of more accurate wastewater loading rates for fine textured soils, determination of the causes of soil clogging, determination of acceptable soil properties to insure adequate treatment is maintained, establishment of suitable design factors for soil absorption fields servicing, determination of suitable wastewater loading regimes and patterns for different soil and site characteristics, and the effects of different construction techniques on the soil.

Although considerable field experience has been gained with selected on-site treatment systems, it would be desirable to have long-term experience with these systems in conjunction with appropriate institutional controls. Long-term performance of these systems is needed to develop confidence among regulatory agencies in the reliability of the alternative and the effectiveness of the institutional arrangement.

SECTION 4

CHARACTERIZATION AND IN-HOUSE ALTERATION OF SMALL WASTE FLOWS

The characteristics of waste flows from residential dwellings, as well as non-residential establishments, can have a profound effect on the performance of individual treatment and final disposal methods. Various water-use events within a dwelling or establishment create an intermittent flow of wastewater which can vary widely in strength and volume. Quantitative and qualitative characterization information is necessary to: 1) provide for the effective design of treatment and disposal systems, 2) facilitate the development of methods to beneficially alter the typical waste characteristics, and 3) facilitate the development of methods to recycle resources in a beneficial manner.

A characterization study was made in three phases. The first phase included the characterization of rural household wastewater. Three objectives were established for this effort: 1) identify the contributions made to the wastewater stream by various individual water-using events within the home, 2) identify the patterns in water usage and hence wastewater production on an hourly, daily and seasonal basis, and 3) identify the qualitative characteristics of the wastewater resulting from the various water-use events. At each study home, efforts were made to monitor water use and/or wastewater production for a sufficient length of time to provide representative, useful characterization information. Water use (wastewater production) was monitored at eleven homes for a total of 434 days. Chemical/physical wastewater characteristics were identified through monitoring at four households for a total of 35 days. The microbiological characteristics of selected household event wastewaters were determined through in-house sampling at each of six households.

The second phase of the study involved the characterization of wastewaters produced by various commercial establishments. A major objective of this phase of the study was to compile a comprehensive summary of the existing, but scattered, characterization data. A second objective was to consider and evaluate various methodologies used for estimating water use/wastewater.

A third phase of the study involved an investigation of methods for in-house alteration of the typical characteristics of small wastewater flows, primarily those from households. Although it was not among the major objectives of this study to actively evaluate methods of in-house alteration, the results of the characterization phases identified areas offering potential for wastewater alteration, and investigation and discussion of this topic was deemed necessary.

The following sections summarize these efforts. More detailed information may be found in Appendix A of this report.

CHARACTERISTICS OF HOUSEHOLD WASTEWATER

Water Use/Wastewater Production

For each event selected for study, the average water use (waste flow) per occurrence and the number of occurrences per capita per day were determined as shown in Tables 1 and 2. The daily flow resulting from each is presented in Table 3, and illustrated in Figure 1.

TABLE 1. WATER USED PER OCCURRENCE, L

Family Unit	Toilet		Laundry		Bath/shower		Dishwasher		Water Softener	
	Mean	S.D.[1]	Mean	S.D.	Mean	S.D.	Mean	S.D.	Mean	S.D.
A	16.6	-	134	7.3	119	62.8	64.3	≈0	286	7.6
B	14.4	-	43	6.0	79	38.1	38.6	16.2	-	-
C	12.5	-	137	10.5	89	53.6	42.3	14.4	271	17.5
D	11.3	-	126	14.3	76	39.2	46.1	14.5	360	42.5
E	18.1	-	158	18.6	72	48.6	43.8	15.0	-	-
F	17.0	-	103	32.3	84	34.7	48.4	14.6	-	-
G	16.3	-	108	38.7	70	59.8	39.7	12.0	264	9.1
H	15.1	-	132	12.9	62	36.2	29.9	8.7	251	6.4
I	17.8	-	105	27.6	80	40.1	47.2	17.8	-	-
J	16.6	-	132	15.3	81	35.1	52.2	10.5	-	-
K	14.0	-	145	8.0	80	30.7	50.3	14.7	547	238
Weighted Average	15.1	-	127	26.7	81	44	47.2	14.7	307	121

[1] In all cases, the standard deviation for toilet flush was computed to be less than 1.5 L.

TABLE 2. FREQUENCY OF OCCURRENCE, uses/cap/day

Family Unit	Toilet	Laundry	Bath/Shower	Dishwasher	Water Softener
A	2.07	0.36	0.43	0.29	0.08
B	2.29	0.19	0.38	0.26	-
C	1.70	0.36	0.31	0.31	0.06
D	2.69	0.23	0.66	0.41	0.02
E	1.71	0.33	0.45	0.24	-
F	1.39	0.46	0.26	0.39	-
G	1.49	0.15	0.47	0.36	0.05
H	2.29	0.32	0.36	0.36	0.24
I	1.68	0.59	0.34	0.49	-
J	31.0	0.27	0.57	0.40	-
K	2.93	0.34	0.55	0.54	0.03
Weighted Average	2.29	0.31	0.47	0.39	0.03

TABLE 3. INDIVIDUAL EVENT WATER USAGE, L/cap/day

Family Unit	Sampling Days	Toilet	Laundry	Bath/ Shower	Dish-washer	Water Softener	Other	Total	90% Confidence Interval
A	28	34.4	47.6	51.0	18.9	23.0	39.3	214	185-245
B	14	32.9	8.3	30.2	10.2	-	15.1	96	76-117
C	77	21.2	49.5	27.6	13.2	15.1	20.4	147	130-163
D	42	31.8	28.7	50.3	18.9	8.7	17.0	155	143-167
E	28	31.0	52.9	32.8	10.6	-	29.9	157	122-192
F	28	23.4	48.0	21.5	18.9	-	15.5	127	105-150
G	35	24.6	15.9	32.9	14.0	12.9	12.5	113	100-125
H	24	34.8	42.3	22.3	11.0	59.3	18.5	188	169-206
I	28	29.9	6.12	26.8	23.0	-	16.6	158	139-177
J	68	52.2	35.9	46.1	20.8	-	15.9	170	162-179
K	62	41.2	49.1	44.2	26.8	17.8	35.9	215	192-239
Weighted Average		34.8	39.7	37.8	18.5	9.8	20.4	161	154-168
% of Daily Flow		21.6%	24.7%	23.5%	11.5%	6.1%	12.7%	100%	-

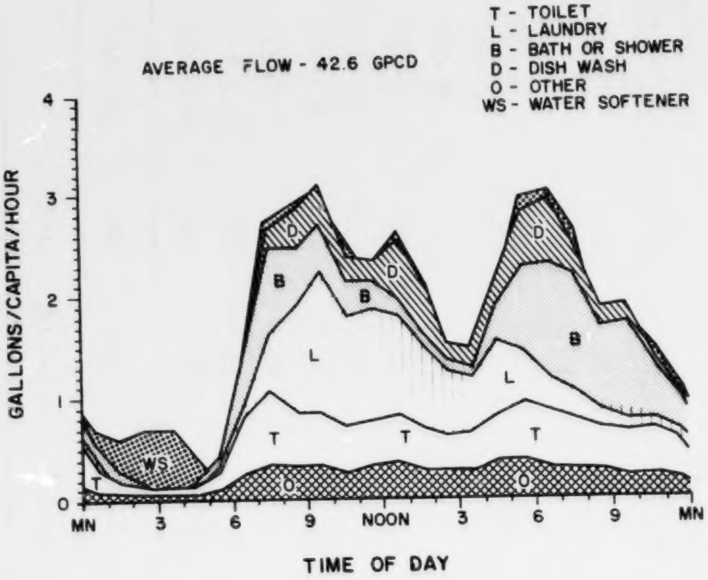


Figure 1. Average daily flow patterns from eleven rural households.

Inspection of these tables reveals a significant variation in water use habits between the homes studied.

The per capita wastewater flow from a single household was 161 L/cap/day (42.6 gal/cap/day) with a 90 percent confidence interval of from 154 to 168 L/cap/day (40.8 to 44.4 gal/cap/day). These daily per capita flow figures are compared with results of previous investigators in Table 4. With the exception of the toilet contributions, there is fair agreement among investigators. The lower toilet contributions found in this study were the result of a lower frequency of useage, likely the result of the use of facilities outside of the home.

It is apparemnt from these studies that major waste flow contributions come from the laundry, bathing and the toilet events. Within the 90 percent confidence level, little day to day variation in flow was found except for laundry and bathing flows, which indicates a relatively consistent daily routine. No significant seasonal effects on in-house water use were detected in these studies. The widely varying characteristics of flow from home to home were more important in determining the flow than the season of the year.

Wastewater Quality

For each event selected for study, the quality characteristics are reported as pollutant contribution per event occurrence and per capita per day. Data, summarized on a mass per capita and a mean pollutant concentration basis for each waste generating event, are presented in Tables 5 and 6. All analyses

TABLE 4. INVESTIGATOR COMPARISON OF EVENT WATER USAGE, L/cap/day

Event	Laak (1975)	Cohen and Wallman (1974)	Ligman, et al. (1974)	Bennett and Linstedt (1975)	This Study
Toilet	74.8	65.0	75.6	55.6	34.8
Bath/Shower	32.1	23.8	47.2	32.9	37.8
Bathroom Sink	7.9		-	18.9	-
Kitchen Sink	13.6	68.0	-	9.8	-
Dishwashing	-		13.2	4.2	18.5
Garbage Grinding	-		5.7	3.0	-
Clothes Washing	28.0	39.7	37.8	43.8	39.7
Water Softener	-	-	-	-	9.8
Miscellaneous	-	-	-	-	20.4
Total	156	197	180	168	161

reported were the result of direct measurement except for mass/cap/day values which included some estimates concerning event frequences.

The daily per capita pollutant contributions determined in this study were compared to the results of previous investigations for BOD_5, suspended solids, nitrogen and phosphorus, as shown in Table 7. Garbage disposal contributions are not included in the results shown. The results are in excellent agreement with the exception of nitrogen. As discussed previously, the lower nitrogen contribution determined in this study was due to a lower frequency of toilet usage. When garbage grinders are used, the contributions of BOD_5 and suspended solids are dramatically increased, but little nitrogen or phosphorus is added (Table 8).

Microbiological analyses were conducted on a selected number of samples collected directly from bathing and clotheswashing events from six study households. Results of these analyses appear in Table 9. It is apparent that a wide range of indicator organisms may be found in these household events. Further characterization of coliform and streptococcal isolates indicated that the major source of bacterial contamination was from the natural environment or skin flora of man.

In occassional instances (1 home in 3; 7 of 47 samples) Pseudomonas aeruginosa was found, but in concentrations below 20/100 mL. Sampling for virus in a home with an infant just receiving an oral polio vaccination produced no viral isolates from the clotheswashing events and detected virus in only one of five bath water samples.

CHARACTERISTICS OF COMMERCIAL WASTEWATER

Characterizing the wastewaters generated by rural commercial establishments and public facilities is a more complex task than for individual homes.

TABLE 5. MEAN POLLUTANT CONCENTRATIONS[1], mg/L

Parameter	Toilet Flush		Garbage Disposal	Kitchen Sink Usage	Automatic Dish Washer	Clotheswasher		Bath/ Shower
	Fecal	Nonfecal				Wash	Rinse	
BOD_5 U	610	330	1030	1460	1040	380	150	170
BOD_5 F	330	200	240	800	650	250	110	100
TOC U	500	220	690	880	600	280	100	100
TOC F	220	160	370	720	390	190	72	61
TS	1500	910	2430	2410	1500	1340	410	250
TVS	1090	610	2270	1710	870	520	180	190
TSS	880	320	1490	720	440	280	120	120
TVSS	720	260	1270	670	370	170	69	85
TOT-N	210	140	60	74	40	21	6	17
NH_3-N	84	27	.9	6	4.5	.7	.4	.2
NO_3-N	.9	1.1	0	.3	.3	.6	.4	.4
TOT-P	38	14	12	74	68	57	21	2
Ortho-P	16	10	8	31	32	15	4	1
Temperature, °F	66	66	71	80	101	90	83	85
Flow[2]	16.3	16.3	14.4	18.1	45.4	59.3	54.4	49.1
Number of Samples	32-40	24-37	4-7	7-11	13-15	24-27	24-28	18-24

[1] Carriage water contributions have been subtracted from the values shown.

[2] Flow values were determined in the wastewater quality study and are in Liters.

TABLE 6. MEAN PER CAPITA POLLUTANT CONTRIBUTIONS[1], mg/cap/day

Parameter	Toilet Flush Fecal	Toilet Flush Nonfecal	Garbage Disposal	Kitchen Sink Usage	Automatic Dish Washer	Clotheswasher Wash	Clotheswasher Rinse	Bath/ Shower
BOD_5 U	4340	6380	10900	8340	12600	10800	4010	3090
BOD_5 F	2340	3980	2570	4580	7840	6970	2840	1870
TOC U	3530	4250	7320	5000	7280	7700	2610	1750
TOC F	1580	3170	3910	4110	4690	5380	1910	1130
TS	10700	17800	25800	13800	18200	37500	10900	4590
TVS	7760	12000	24000	9730	10500	14700	4800	3600
TSS	6240	6280	15800	4110	5270	7930	3040	2260
TVSS	5090	5120	13500	3840	4460	4700	1810	1580
TOT-N	1500	2640	630	420	490	580	150	310
NH_3-N	590	520	9.6	32.3	54	19.4	11.4	40
NO_3-N	6.3	21.1	.2	1.8	4.1	17	10.3	7.4
TOT-P	270	280	130	420	820	1600	550	36
Ortho-P	120	190	90	180	380	410	110	20

[1] Carriage water contributions have been subtracted from the values shown.

TABLE 7. DAILY POLLUTANT CONTRIBUTIONS[1], gram/cap/day

Pollutant	Olsson, et al. (1968)	Ligman et al. (1974)	Laak (1975)	Bennett and Linstedt (1975)	This Study
BOD_5	35.0	48.1	48.6	34.8	49.6
Suspended Solids	48.0	46.3	-	47.3	35.1
Nitrogen	12.1	16.8	-	7.2	6.1
Phosphorus	3.8	4.1	-	-	4.0

[1] Garbage disposal contributions have been excluded.

TABLE 8. INCREASE IN POLLUTANT MASS DUE TO GARBAGE DISPOSALS, grams/cap/day

Pollutant	Ligman, et al. (1974)	Bennett and Linstedt (1975)	This Study
BOD_5	30.9 (64%)[1]	12.3 (35%)	10.9 (22%)
Suspended Solids	43.6 (94%)	20.2 (43%)	15.8 (45%)
Nitrogen	.9 (5%)	.2 (3%)	.6 (10%)
Phosphorus	-	.1	.1

[1] Percentage increase of the corresponding value shown in Table 7.

This is principally due to the fact that these establishments serve transient populations.

As part of this study, a detailed literature review was made for a variety of commercial establishments, and a limited sampling effort was conducted. A summary of the quantitative wastewater characteristics determined from the literature review and the limited sampling, is presented in Table 10. Details regarding this characterization effort are presented in Appendix A.

IN-HOUSE ALTERATION OF HOUSEHOLD WASTEWATER CHARACTERISTICS

The characteristics of a given wastewater exert a profound influence in determining the pollution control strategy required for its effective management. Recognizing this fact, efforts are currently being made to develop methods for beneficially altering the typical wastewater characteristics through source manipulation. Elimination or the isolation in a concentrated waste stream of potential pollutants at the source, such as flow, BOD_5, suspended solids, nutrients and pathogenic organisms may enhance

TABLE 9. BACTERIOLOGICAL CHARACTERISTICS OF BATH AND LAUNDRY WASTEWATERS

Event		Samples	#/100 mL	Confidence Intervals,[1] #/100 mL	
				95%	99%
Clothes Washing[2]	Total Coliforms	41	215	65-700	45-1020
	Fecal Coliforms	41	107	39-295	28-405
	Fecal Streptococci	41	77	27-220	19-305
Bathing	Total Coliforms	32	1810	710-4600	530-6160
	Fecal Coliforms	32	1210	450-3240	330-4410
	Fecal Streptococci	32	326	100-1050	70-1510

[1] Log-normalized data.

[2] Samples were obtained from the middle of the wash cycle. Samples taken from 15 rinse cycles were consistently lower than the corresponding wash cycle values.

conventional disposal methods or may facilitate the development of innovative waste management schemes. It should be emphasized that any techniques used to affect this alteration could be directed toward commercial establishments as well as residential dwellings.

Two techniques for altering wastewater characteristics are: 1) waste flow reduction, and 2) waste segregation. A brief synopsis of the use of these strategies is presented below. A detailed description and discussion of these methods appear in Appendix A.

Waste Flow Reduction

Nearly 70 percent of the total wastewater generated in the homes is derived from the toilet, laundry and bath (Witt, et al. 1974). The most substantial water savings, therefore, can be made in these areas. Low flow toilets, "sudsaver" washing [illegible]nes, restricted flow shower heads, and recycling of bath and laundry wastes for toilet flushing are four commonly mentioned water saving devices. By reducing the toilet flushing volume to 3 gallons, clothes washing to 28 gallons by using a sudsaver, and baths and showers to 15 gallons, average water use could be reduced to 17 percent in rural Wisconsin homes. Recycling bath and laundry wastes to flush toilets could increase the savings to 33 percent (Table 11). These savings compare with values from other studies (Cohen and Wallman, 1974; Bishop, 1975).

Waste Segregation

The characterization studies indicated that the individual wastewater contributing events may be divided into three major categories: 1) garbage disposal wastes, 2) toilet wastes, and 3) basin, sink and appliance wastewaters.

TABLE 10. QUANTITATIVE WASTEWATER CHARACTERISTICS SUMMARY

Establishment Category	Unit	Range of Mean Daily (No. of Data Pts.)	Maximum Day / Mean Day	Peak Hour / Mean Day
Bar/Tavern	Seat	5-15 (1)[2]	-	-
Bowling Alleys	Alley	49-56 (1)[2]	-	-
Campgrounds	Camper	40 (1)	-	-
Picnic Parks	Picnicker	5 (1)	-	-
Churches	Member	.20 (3)	4.4	24.0
Golf Clubs	Golfer	4-16.9 (1)[3]	1.2	1.9[2]
Laundromats	Machine	170-240 (3)	3.0	-
Motels	Patron Space	70	2.1	4.5,6.9
Restaurants	Meal	6.4-11.7 (3)	1.4	2.0
Schools I[1]	Pupil	5-6 (2)	2.4	-
Schools II[1]	Pupil	6.2-7.5 (4)	1.8,1.4,1.1	3.8
Schools III[1]	Pupil	8.2-19.9 (4)	1.1,2.3,3.1	
Service Stations	Bay	290-600 (4)	3.7	26.5
Shopping Centers	1000 ft^2	164-215	1.4	2.0,2.0,2.5
Sports Facilities	Spectator	1.2[2]	-	7.5
Theaters	Seat	2.7[2]	-	-

[1] School categories: I - restrooms only, II - restrooms and cafeteria, III - restrooms, cafeteria and showers.

[2] Verification desirable.

[3] High value due to 700 seat restaurant contributions.

TABLE 11. PROJECTED FLOW REDUCTION SUMMARY

		L/cap/day with Method Shown				% Reductions		
Family Unit	Mean Meas. L/cap/day	(1) 11 L/Flush Toilet	(2) Sudsaver-Washer-105 L/use	(3) 57 L Bath/ Shower	(4) Recycle Systems	Using (1)-(3)	Using (4)	Using (1)-(4)
A	214	204	205	188	191	22	11	33
B	96	89	96	88	70	16	27	43
C	147	145	135	137	128	16	13	29
D	155	155	151	142	124	11	21	32
E	157	145	139	150	137	23	13	36
F	128	120	128	121	112	12	12	24
G	113	105	113	106	96	7	15	22
H	188	179	179	186	162	10	14	24
I	158	147	158	150	139	11	12	24
J	171	154	163	156	135	23	19	43
K	215	207	202	202	182	16	15	31
Average (Weighted)	161	152	154	150	135	17	16	33

A summary of selected chemical/physical characteristics of each fraction as determined in this study and by previous investigations is presented in Table 12.

Elimination of the garbage disposal and the use of an alternative toilet system, such as a composting or closed-loop recycle system, offers the potential to substantially reduce the flow volume and pollutant load of residential households to that of the sink, basin and appliance wastewaters, collectively referred to as grey water. Further, microbiological studies conducted as part of the characterization effort described previously, indicated that household grey water possess a significantly reduced potential for harboring pathogenic organisms as compared to the toilet waste or the household wastewater as a whole.

Very little research has been conducted to identify the operation and performance of various strategies for managing the separated waste streams. As part of this study, a laboratory investigation of the treatment of grey water by septic tank - sand filtration was conducted. A discussion of this research has been presented in Section 5 of this report with greater detail given in Appendix A.

TABLE 12. AVERAGE POLLUTANT CONTRIBUTIONS OF MAJOR RESIDENTIAL WASTEWATER FRACTIONS, grams/capita/day

Fraction	Garbage Disposal	Toilet	Basins, Sinks, Appliances	Approximate Total Contributions
BOD_5	18.0[1] 10.9-30.9[2] (1,2,3)[3]	16.7 6.9-23.6 (1,2,3,4,6)	28.5 24.5-38.8 (1,2,3,4,6)	63.2
Suspended Solids	26.5 15.8-43.6 (1,2,3)	27.0 12.5-36.5 (1,2,3,4)	17.2 10.8-22.6 (1,2,3,4)	70.7
Nitrogen	0.6 0.2-0.9 (1,2,3)	8.7 4.1-16.8 (1,2,3,4)	1.9 1.1-2.0 (1,2,4)	11.2
Phosphorus	0.1 0.1-0.1 (1,2)	1.2 0.6-1.6 (1,3,4)	2.8 2.2-3.4 (1,3,4)	4.0
Approximate Flow gal/c/d	2 (1,2,3)	16 (1-6)	29 (1-6)	47

[1] Mean of Study Average Values

[2] Range of Study Average Values

[3] References used in mean and range calculations as follows:

1. Siegrist et al. 1976.
2. Bennett and Linstedt 1975.
3. Ligman et al. 1974.
4. Olsson et al. 1968
5. Cohen and Wallman 1974.
6. Laak 1975.

SECTION 5

UNIT PROCESSES FOR TREATMENT OF SMALL WASTEWATER FLOWS

An important step in the handling of wastewaters for ultimate disposal onsite, is the pretreatment process. The quality of wastewater discharged to the environment is dependent upon the environmental objectives of the locality. It is likely that high quality restrictions will be placed upon wastewater discharging directly to surface waters although similar requirements may be placed upon direct discharges to subsurface waters where sufficient soil depth is not available (e.g., discharges to creviced bedrock underlying a thin soil mantle). Less stringent quality requirements prevail for wastewaters being evapotranspired or being discharged to the soil mantle which in itself acts as a treatment process.

There are numerous alternative processes currently available to treat small flow wastewaters. A tabulation of some of the process alternatives available appear in Table 13. It should be noted that soil is considered a most important treatment process and will be considered separately under Section 6.

Not all potential processes for treatment were examined in this study but a brief review of the status of many of the potential processes is presented in Appendix A. This study evaluated a selected number of unit processes under both laboratory and field conditions. Details of these studies also appear in Appendix A. A brief synopsis of these findings is discussed below.

BIOLOGICAL PROCESSES

Anaerobic Processes

Anaerobic treatment processes involve the decomposition of organic or inorganic materials in the absence of dissolved oxygen. These processes have normally been restricted to the treatment of highly concentrated wastewaters and sludges but have recently found useful applications in the denitrification of highly nitrified wastewaters as well. In small flows treatment applications, septic tanks are the most widely used anaerobic process (although it is clear that anaerobic treatment is only one function of the septic tanks), but some work has been reported on fixed film anaerobic contactors for organic stabilization as well as denitrification.

Septic Tanks--

The septic tank serves several important functions in wastewater treatment including solid-liquid separation, storage of solids and floatable materials, and anaerobic treatment of both stored solids as well as non-settleable

TABLE 13. SELECTED TREATMENT PROCESS ALTERNATIVES

- In-House
  - Water Conservation
  - Waste Segregation
- Biological Processes
  - Fixed Film - Aerobic/Anaerobic
  - Suspended Growth - Aerobic/Anaerobic
- Physical Chemical Processes
  - Adsorption
  - Chemical Precipitation/Coagulation
  - Disinfection
  - Ion Exchange
- Soil Mantle

materials. Details of design requirements for septic tank systems can be found elsewhere (Appendix A).

The results from monitoring seven field septic tank installations are summarized in Table 14. As would be expected for this type of process, effluent quality was relatively poor with respect to organic matter, indicator organisms, nitrogen and phosphorus. Suspended solids values were indicative of adequate solids separation.

The following observations were also noted.

1. There were significant differences on the basis of 95 percent confidence intervals in the average BOD_5 and COD levels between septic tanks studied, ranging from 57 mg/L (site J) to 272 mg/L (site C) for BOD_5 and 208 mg/L (site J) to 542 mg/L (site C) for COD.

2. For most sites, there was not a significant difference in the total suspended solids (TSS) levels between units (site J had significantly less TSS than sites A and C). The average concentration was 49 mg/L for all units as a group with range of averages from 34 mg/L (site J) to 69 mg/L (site A).

3. The level of total nitrogen varied considerably between units (from 26 mg/L at site E to 76 mg/L at site C). When considering

TABLE 14. COMPARISON OF SEPTIC TANK AND AEROBIC UNIT EFFLUENTS

Sites and Dates	Aerobic Units C, G, H, J[1]	Septic Tanks- A, B, C, D, E, F, J[1]
Parameter and Statistics	Jul 1972- Dec 1976	May 1972- Dec 1976
BOD_5 (unfiltered), mg/L		
Mean (# of Samples)	37(112)	138(150)
Coeff. of Variation	0.78	0.42
95% Conf. Int.	32-42	129-147
Range	2-208	7-480
BOD_5 (filtered), mg/L		
Mean (# of Samples)	15(94)	109(130)
Coeff. of Variation	0.94	0.47
95% Conf. Int.	12-18	100-118
Range	1-120	7-330
COD, mg/L		
Mean (# of Samples)	108(116)	327(152)
Coeff. of Variation	0.40	0.33
95% Conf. Int.	100-116	310-344
Range	20-349	25-780
Total Phosphorus, mg-P/L		
Mean (# of Samples)	26(80)	13(99)
Coeff. of Variation	0.74	0.34
95% Conf. Int.	22-30	12-14
Range	6-140	0.7-90
Orthophosphorus, mg-P/L		
Mean (# of Samples)	21(78)	11(89)
Coeff. of Variation	0.54	0.36
95% Conf. Int.	18-24	10-12
Range	6-51	3-20
Fecal Coliforms, $\log_{10}$#/L		
Mean (# of Samples)	5.0(115)	6.7(151)
Coeff. of Variation	0.30	0.24
95% Conf. Int.	4.7-5.3	6.4-7.0
Range	2.8-7.3	2.0-8.2
Fecal Strep, $\log_{10}$#/L		
Mean (# of Samples)	4.3(113)	4.6(155)
Coeff. of Variation	0.49	1.0
95% Conf. Int.	3.9-4.7	3.9-5.3
Range	2.0-6.3	2.0-7.6
Suspended Solids, mg/L		
Mean (# of Samples)	39(117)	49(148)
Coeff. of Variation	(.23)	0.16
95% Conf. Int.	33-46	44-54
Range	3-252	10-695
Volatile Suspended, mg/L		
Mean (# of Samples)	27(118)	35(148)
Coeff. of Variation	.26	0.18
95% Conf. Int.	23-32	32-39
Range	1-144	5-320
Total Nitrogen, mg-N/L		
Mean (# of Samples)	36(87)	45(99)
Coeff. of Variation	0.32	0.40
95% Conf. Int.	34-38	41-49
Range	15-78	9-125
Ammonia Nitrogen, mg-N/L		
Mean (# of Samples)	0.9(92)	31(108)
Coeff. of Variation	4.2	0.46
95% Conf. Int.	0.1-1.7	28-34
Range	<0.1-60	0.1-91
Nitrate Nitrogen, mg-N/L		
Mean (# of Samples)	30(95)	0.4(114)
Coeff. of Variation	0.44	6.7
95% Conf. Int.	27-33	<0.1-0.9
Range	0.3-72	<0.1-74

[1] Letters designate field sites; See Appendix A.

all the septic tanks as a group, 69 percent of the nitrogen was in the form of ammonia (ranged from 60 percent at site C to 82 percent at site D).

4. There was essentially no difference in the total phosphorus concentrations between the seven septic tanks studies. Approximately 85 percent of the phosphorus was in the form of orthophosphate.

5. There was no difference in the levels of indicator bacteria isolated from the field units. The average fecal coliform level was 5×10^6 organisms per liter, whereas the fecal streptococci count was 4×10^4 organisms per liter.

A comparison of the findings of this study with those reported by others is presented in Table 15. In reviewing the results of the various research studies, it appears that the suspended solids concentration in the effluent was related to: 1) size of tank, 2) degree of compartmentalization, and 3) frequency of pumping. The largest tanks reported in Table 15 were installed as part of this study and at the University of Connecticut (Laak, 1973). The beneficial effect of compartmentalization has been adequately demonstrated by others (Ludwig, 1950; Weibel, et al., 1954), as well as in this study. As solids build up within a septic tank, the chances for solids washout increase. In studies where sludge build-up was accelerated (Weibel, et al., 1949; 1954) a deterioration of effluent quality occurred once the solids holding capacity had been exceeded. None of the septic tanks in this study were monitored long enough to note this phenomenon.

Biological Denitrification Processes--

Nitrogen removal from wastewaters may be of interest when ammonia or nitrate limitations are imposed on certain receiving waters. Biological denitrification processes have been shown to be practical in large scale limitations, but long term experience is needed. Application of this concept to low flow systems has not been extensively tested outside of the laboratory. Several alternative schemes have been tested in this study and are reported in Appendix A. One field installation has also produced some preliminary data. Denitrification was achieved over a three-month period in an upflow system employing approximately 0.75 m^3 of 0.9 cm dolomite stone. Well nitrified sand filtered effluent was dosed with methanol, and then pumped to the dolomite bed. After a retention time ranging from 24 to 36 hours, nitrate levels were reduced from an average of 34 mg/L N to 3.7/mg/L N with very little residual methanol.

Aerobic Processes

Aerobic processes involve biochemical transformations of organic and inorganic materials in the presence of dissolved oxygen. Due to the characteristics of aerobic processes, effluent qualities are normally superior to those of anaerobic systems. Over 75 years of experience with this biological process in large scale applications makes it a logical candidate for small flows treatment. Numerous generic types of aerobic processes have been developed for small flows applications including suspended and fixed film systems.

TABLE 15. SUMMARY OF EFFLUENT DATA OF VARIOUS SEPTIC TANK STUDIES

Site, Dates & Source / Parameter and Statistics	This Study	Cincinnati	New York	Toronto	Cincinnati	Connecticut
	1972-1976	1947-1948	1955	1961-1966	1969	1972
	7 sites	10 tanks	19 sites	4 sites	1 site	1 tank
	(Table A-113)	(Weibel, 1949)	(Salvato, 1955)	(Bernhart, 1967)	(Thomas & Ben-dixen, 1969)	(Laak, 1973)
BOD_5, mg/L						
Mean (# of Samples)	138(150)	138(44)*	140(51)	240(21)†	93	120(50)
Coeff. of Variation	0.42	0.35		0.06		0.48
95% Conf. Int.	129-147	-		210-280		100-130
Range	7-480	64-256		70-385		30-280
COD, mg/L						
Mean (# of Samples)	327(152)				220	200(50)
Coeff. of Variation	0.33					0.41
95% Conf. Int.	310-344					175-225
Range	25-780					71-360
Suspended Solids, mg/L						
Mean (# of Samples)	49(148)†	155(55)*	101(51)	95(18)†	45	39(47)
Coeff. of Variation	0.16	0.54		0.08		0.98
95% Conf. Int.	44-54	-		79-115		28-50
Range	10-695	43-485		48-340		8-270
Total Nitrogen, mg/L						
Mean (# of Samples)	45(99)		36(51)		33	
Coeff. of Variation	0.40					
95% Conf. Int.	41-49					
Range	9-125					
Volume, m^3						
Mean (# of Samples)	4.7(7)	2.6(10)	-	≈ 1.8	not reported	4.0(1)
Range	3.5-7.6	1.7-7.8				

* Calculated from the average values from 10 tanks, 6 series of tests.

† Calculated on the basis of a log-normal distribution of data.

These range from the simple lagoon to the more complex extended aeration process.

Mechanical propietary units--Field and laboratory studies conducted as a portion of this project have dealt with propietary mechanical devices available on the market for small flows applications. A detailed description of the processes studied, their operation and maintenance requirements and their performance characteristics is found in Appendix A.

The results of studies of four field installations employing proprietary aerobic processes are presented in Table 14. Also included is a summary of the grouped septic tank data for comparison. It is apparent that, in the aerobic units studied, a higher degree of treatment was achieved as compared to septic tanks aut that periodic malfunctions and upsets resulted in substantially higher variability in the effluent quality (Otis and Boyle, 1976). Note that effluent suspended solids concentrations from the aerobic units were not significantly different for septic tanks at the 95 percent confidence interval.

The following observations concerning the aerobic units were also noted:

1. Because of solid-liquid separation difficulties owing to a variety of operational difficulties, there was considerable difference in the average TSS levels between the various units, ranging from 12 mg/L to 65 mg/L.

2. The total nitrogen concentrations did not vary between sites as the septic tanks did. When using the grouped data, 83 percent of the total nitrogen was in the form of nitrate.

3. One unit which received septic tank effluent (see Table A-113) and employed cycled aeration, removed over 1/2 of the total nitrogen. Another field unit employing continuous aeration also received septic tank effluent but only removed about 15 percent of the total nitrogen.

4. The total phosphorus levels varied substantially from site to site since phosphorus loading was generally a function of household laundry use (Witt, et al., 1974) which varied from home to home. About 18 percent of the phosphorus was in the form of orthophosphate.

5. Except for one system which was lightly loaded, there was essentailly no difference in the bacterial counts between aerobic units. There was also no difference between the fecal streptococci levels from the aerobic units versus the septic tank. The fecal coliform levels were almost 2 orders of magnitude lower in the aerobic units than in the septic tanks (10^5 versus $10^{6.7}$ per liter).

6. As indicated by the coefficient of variation, the aerobic units produced a more variable effluent quality in terms of organic matter and suspended solids.

Periodic carryover of solids was the major reason for effluent quality deterioration from the aerobic units studied (Otis and Boyle, 1976). Bulking sludge, excessive sludge build up, poor return of both underflow solids and scum from the clarifier and plugged or inoperative lines seemed to be the most common causes of carryover. Detailed discussions of operation and maintenance characteristics of both laboratory and field installations are presented in Appendix A. It is clear, however, that proper installation and regular servicing and inspection is necessary to insure proper performance. Inspections should be performed at least every two months and excess solids pumped every eight to twelve months.

Figures 2 and 3 and Table 16 illustrate the results of this study on both septic tanks and aerobic units as compared with other investigatiors. Again, wide variations in effluent quality were reported for aerobic units and other investigators also reported serious difficulties with effluent solids overflow. In most cases, investigators pointed to improper maintenance and supervision as reasons for poor unit performance (Bennett and Linstedt, 1975; Glasser, 1974; Voell and Vance, 1974).

Intermittent filtration--Field experience to date with on-site treatment processes has indicated that further polishing of effluents will be necessary in many cases in order to meet environmental quality objectives. Filtration appears to be one of the most promising alternatives currently available to provide this polishing step. Whether the filtration is provided by granular beds or by mechanical filter systems employed as a part of the biological process or as a separate process, depends upon economics, effectiveness and maintenance requirements.

Granular filtrations appears to be particularly well suited to on-site system design. At least three basic flow configurations have been successfully tested in the field; the buried filter, the recirculating sand filter and the intermittent sand filter.

The buried sand filter is constructed below the soil surface. A bed is excavated and underdrain collectors are installed. Approximately 30 cm (1 ft) of gravel are then placed over the bottom. Normally, 60 cm (2 ft) of sand (usually greater than 0.4 mm effective size with a uniformity coefficient of less than 4.0) is then placed over the gravel, followed by influent drain tile placed in 25 to 30 cm (10 to 12 in) of gravel. This bed is subsequently covered with at least 15 cm (6 in) of topsoil. Allowable wastewater loadings vary from state to state, but range between .03 to .06 m/day (0.75 and 1.5 gpd/ft^2) depending upon appliance loading and pretreatment. Performance of these systems are normally excellent, unless overloaded, but inaccessability of the bed dictates a more conservative sizing than might be otherwise employed for open systems.

The recirculating sand filter system consists of a septic tank, a recirculation tank, and an open sand filter (Hines and Favereau, 1974). Wastewater is dosed on to the filter by a submersible pump located in the recirculation tank. The sump pump is actuated by a time clock and is sized to pump between 19 to 38 L/min (5 to 10 gal/min) for single households. A recirculation ratio of 4:1 (recycle to forward flow) is recommended. The recirculation

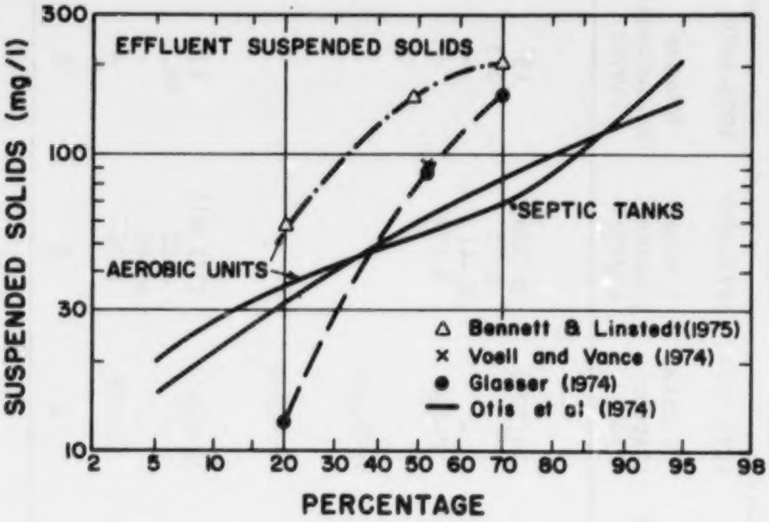


Figure 2. Comparisons of septic tank and aerobic unit effluent suspended solids.

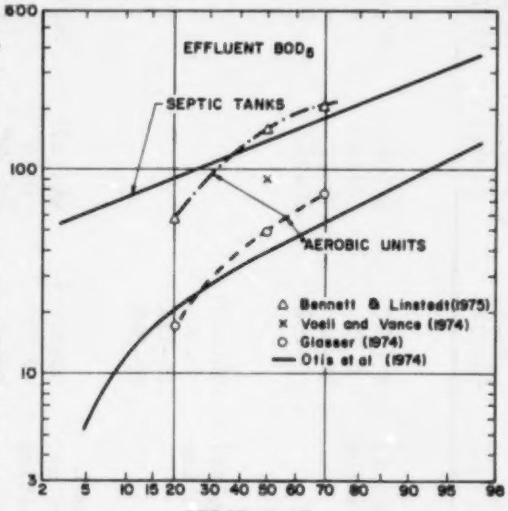


Figure 3. Comparisons of septic tank and aerobic unit effluent BOD_5.

TABLE 16. SUMMARY OF EFFLUENT DATA OF VARIOUS AEROBIC TREATMENT UNIT STUDIES

Parameter and Statistics / Site, Dates & Source	This Study	Toronto	New York	Colorado	Colorado	Maryland	NSF
	1972-1976	1965-1967	1972-1973	-	1971-1974	1973-1974	1971-1974
	4 Sites (Table) (A-114)	5 Sites (Bernhart) (1967)	146 Sites (Voell) (1974)	- (Bennett) (1975)	55 Sites (Tipton) (1975)	5 Sites (Glasser) (1975)	8 Units (McClelland) (1976)
BOD_5, mg/L							
Mean (# of Samples)	37(112)	47(86)[1]	92(146)	150(16)	144(393)	36(124)[1]	13[2]
Coeff. of Variation	0.78	0.13	-	≈0.8	-	0.21	≈0.3
95% Conf. Int.	32-42	42-52	-	-	-	31-41	-
Range	0-208	10-280	-	≈10-300	10-824	3-170	-
COD, mg/L							
Mean (# of Samples)	108(116)				385(108)		
Coeff. of Variation	0.40				-		
95% Conf. Int.	100-116				-		
Range	20-349				88-1030		
Suspended Solids, mg/L							
Mean (# of Samples)	39(117)[1]	94(74)[1]	94(146)	150(16)	122(251)	57(132)[1]	17[2]
Coeff. of Variation	0.23	0.13	-	≈0.6	-	0.22	≈0.2
95% Conf. Int.	33-46	82-108	-	-	-	49-66	-
Range	3-252	18-692	-	≈50-300	17-768	4-366	-
Number of Unit Types	4	4	3	-	6	5	8

[1] Calculated on the basis of a log-normal distribution of data.

[2] Determined graphically.

tank, normally the same size as the septic tank, receives flow from the septic tank and the recirculated portion of the filter effluent. Baffles provide proper mixing of the septic tank and filter effluents prior to recycle. Filter effluent recycle flow is controlled by a rubber float valve located in the filter effluent return line. When the recirculation tank is filled, filter effluent is discharged from the system.

The filter bed consists of 90 cm (3 ft) of coarse filter sand with a desired effective size of 0.6 to 1.5 mm and a uniformity coefficient of less than 2.5. Approximately 30 cm (1 ft) of graded gravel support the sand and surround the underdrain system. The filter is designed to operate at a flow rate of 0.2 m/day (3 gpd/ft^2) based on raw septic tank flow. It is estimated that approximately 2 cm (1 in) of sand should be removed once per year in order to avoid serious ponding conditions. After 30 cm (1 ft) of sand have been removed, new sand would be added.

Results from a household system indicate that effluent BOD_5 values average less than 5 mg/L and TSS values less than 6 mg/L BOD_5 (Hines and Favereau, 1974).

With the intermittent granular filter, pretreated wastewater is applied over a 60 to 90 cm (2 to 3 ft) deep bed of sand and the filtrate collected by underdrains. The sand remains aerobic and serves as a biological filter, removing not only suspended solids, but also dissolved organics. A summary of intermittent sand filter performance based upon a review of the literature is given in Appendix A.

In this study, intermittent sand filters receiving septic tank or aerobic unit effluent have been tested under field and laboratory conditions. Since coarse sands were not readily available in Wisconsin as reasonable costs, filters were constructed employing locally available finer sands. Of major concern in sizing of these filters was the tradeoff between effluent quality and maintenance requirements. Figure 4 depicts the relationship that has been found between effluent quality, maintenance requirements, sand size and application rates.

Details of the laboratory and field studies on intermittent sand filters are presented in Appendix A. Effluent quality of sand filtered septic tank and aerobic unit effluents appear in Tables 17 and 18. It may be noted that relatively little difference was found between aerobic unit-sand filter effluent and septic tank-sand filter effluent for comparable loading conditions, although the aerobic unit system employed a finer sand (0.19 mm as compared with 0.45 mm).

Filter runs were dependent upon grain size, hydraulic loading, influent organic strength, and maintenance techniques (Sauer, 1975; Sauer, et al., 1976).

Recommended filter operation schedules for a septic tank-sand filter system are presented in Table 19. In treating septic tank effluent, it is recommended that two filters be employed in an alternating mode, each designed for a hydraulic loading rate of 0.2 m/day (5 gpd/ft^2). When one filter becomes

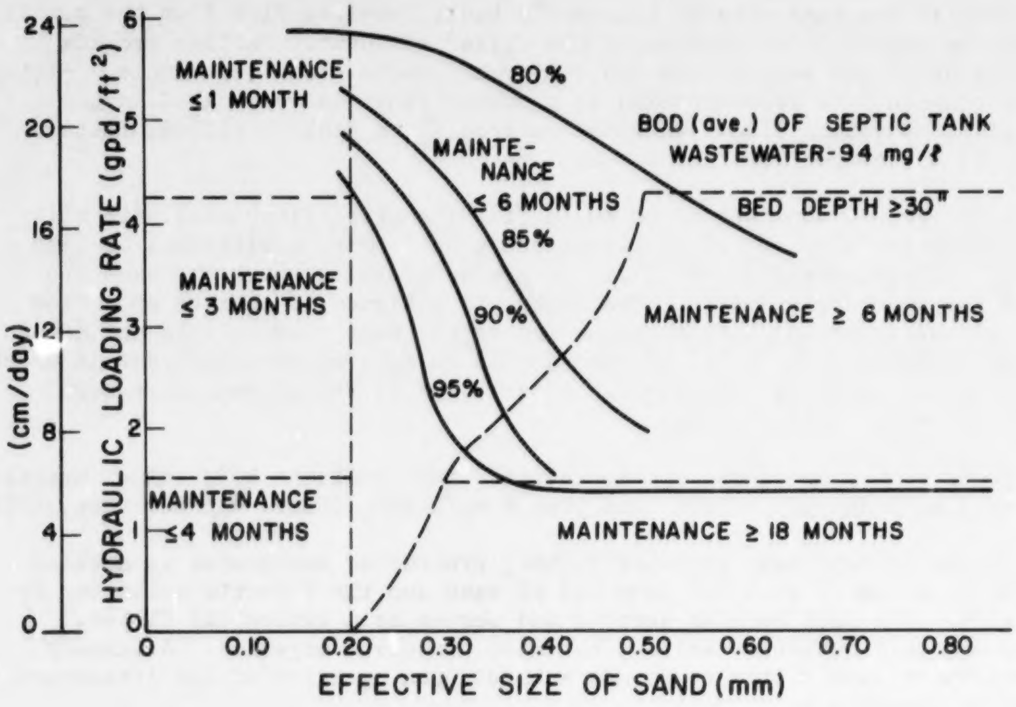


Figure 4. Trends of percent BOD_5 reduction and required maintenance of sand filters reported in the literature treating septic tank wastewater (ave. BOD_5) = 94 mg(L) (Sauer, 1975).

ponded, it is taken out of service, allowed to drain, raked to a depth of 5 to 10 cm (2 to 4 in), and rested prior to reapplication of wastewater. After a second loading period, the top 10 cm (4 in) of sand from that filter should be replaced with clean sand.

Aerobic unit-sand filter systems do not require a second filter. An application rate of 0.2 m/day (5 gpd/ft^2) is suggested with a six-month maintenance interval. Removal of the solids mat, along with 2 cm (1 in) of sand, and replacement with 2 cm (1 in) of clean sand is the only required maintenance step. Reapplication of wastewater is possible immediately after maintenance is performed. Experience has shown that periodic biological and hydraulic upsets of the biological process can be assimilated by the sand filters, however, extended periods of upset will lead to shorter filter runs.

Grey Water Filtration--

Grey water treatment studies to date are preliminary and are based primarily upon sand filtration studies employing the wastewater simulator used for pilot-scale combined wastewater studies (Appendix A). Eight sand filters

TABLE 17. SEPTIC TANK-SAND FILTER EFFLUENT QUALITY DATA (SITE E)

	Septic Tank Effluent	Sand Filter[1] Effluent	Chlorinated Effluent
BOD (mg/L)	116	8	5
TSS (mg/LO	43	4	6
Total Nitrogen-N (mg/L)	26	28	20
Ammonia-N (mg/L)	20	3	2
Nitrate-N (mg/L)	0.2	25	19
Total Phosphorus-P (mg/L)	11	10	8
Orthophosphorus-P (mg/L) (log$_{10}$/liter)	9	8	8
Fecal Coliforms (log$_{10}$/liter)	6.6	3.9	0.3
Fecal Streptococci	4.7	2.4	0.5

[1] Loading rate average: 5 gpd/ft^2 (0.2 m/day).
Effective size--0.45 mm; uniformity coefficient--3.0.

TABLE 18. AEROBIC UNIT-SAND FILTER EFFLUENT QUALITY DATA (SITE H)

	Aerobic Units Effluent	Sand Filter[1] Effluent	Chlorinated Effluent
BOD$_5$ (mg/L)	31	3.5	4
TSS (mg/L)	41	9.4	7
Total Nitrogen-N (mg/L)	37.8	34.8	38.3
Ammonia-N (mg/L)	1.4	0.3	0.4
Nitrate-N (mg/L)	32.3	33.8	37.6
Total Phosphorus-P (mg/L)	29.5	20.3	24.0
Orthophosphate-P (mg/L)	25.0	18.9	23.4
Fecal Coliforms (log$_{10}$/L)	5.3	4.0	0.9
Fecal Streptococci (log$_{10}$/L)	4.4	3.2	1.5

[1] Loading rate average: 3.8 gpd/ft^2 (0.15 m/day).
Effective size--0.19 mm; uniformity coefficient--3.31.

TABLE 19. SEPTIC TANK-SAND FILTER OPERATION SCHEDULE

Sand		
Effective Size (mm)	Uniformity Coefficient	Loading and Resting Period (Months)
0.2	3-4	1
0.4	3	3
0.6	1.4	5

approximately .09 m^2 (1 ft^2) receiving septic tank effluent were operated until hydraulic conductivities decreased to a point where ponding exceeded 25 cm (10 in). Results of these filter studies appear in Tables 20 and 21. Effective sand size was 0.28-0.30 mm with a uniformity coefficient of approximately 2.9 to 3.2. Preliminary results indicated that filter runs with grey water were over twice as long and processed twice as much wastewater as did filters receiving combined household waste. (Based upon a loading of 30 cm/day (7.2 gpd/ft^2) administered in 4 to 5 doses per day.) A distinct difference in clogging matrix was noted both on the surface and in-depth between sand filters receiving grey water and those treating combined household wastewater. Intermittent sand filtration of septic tank treated grey water through 60 cm of sand at a rate of 15 to 30 cm/day (3.6 to 7.2 gpd/ft^2) produced well-nitrified effluents with low BOD_5 and suspended solids.

PHYSICAL CHEMICAL PROCESSES

Numerous physical-chemical processes may find application in the treatment of low flow wastewaters. A discussion of the status of a selected number of these potential systems appears in Appendix A. Among the processes which have been examined during this study are phosphorus precipitation, ammonia ion exchange, and wastewater disinfection.

Phosphorus Removal

Traditional methods for removing phosphorus from effluents involve the addition of coagulating agents such as alum, iron salts or lime. Chemical feed of these salts to small flow systems may abound in maintenance problems and will result in significant volumes of sludge to be handled. More desirable would be packed bed systems, whereby the waste would flow past a fixed coagulating media. Laboratory studies, detailed in Appendix A, indicate that dolomite or calcite packed columns provide limited removal of phosphorus from septic tank effluents. This decrease in sorptive capacity may be due to organic anions in the effluent competing for sorption sites as well as microbial growth blocking the sorption sites. Similar findings were apparent in one field test (Sikora, et al., 1976).

TABLE 20. SAND FILTER OPERATION

Filter No.	Effluent Applied	Application Mode		Run Days to-		Loading/m^2		
		Doses/Day	cm/day	Ponding	Failure	Liters	Grams BOD$_5$	Grams SS
1	Grey	5	29[1] 26-32	226	264	6610	4410	3230
2	Grey	5	29 26-32	260	285	7110	4740	3550
7	Combined	6	31 28-33	104	124	3330	1940	2040
8	Combined	6	31 28-33	108	124	3310	1940	2040

[1] Mean over 95% Confidence Interval for run length.

TABLE 21. SAND FILTER EFFLUENT CHARACTERISTICS

Parameter	Grey Water				Combined Wastewater			
	1	2	3	4	5	6	7	8
Loading cm/day	30	30	15	15	15	15	30	30
BOD$_5$	1(17)[1] 1-3	1(18) 1-3	1(17) 1-3	1(18) 1-2	2(19) 1-3	1(13) 1-3	4(15) 2-7	4(20) 2-6
COD	26(6) 13-39	17(7) 7-27	21(7) 7-35	16(7) 12-20	16(9) 10-23	18(9) 10-25	25(6) 9-40	57(9) 29-86
SS[2]	12(19) 9-16	14(19) 10-19	11(16) 7-16	8(20) 6-10	8(20) 5-12	15(13) 9-23	18(14) 11-31	17(20) 12-25
VSS	8(19) 6-9	7(19) 5-9	7(16) 5-9	5(20) 4-6	5(20) 3-6	6(13) 4-9	10(15) 6-14	8(20) 5-10

[1] Mean (no. of samples); 95% confidence interval.

[2] Log-normalized data.

Nitrogen Removal

Ammonia removal from septic tank effluents employing clinoptilolite were studied in laboratory columns. There is an abundance of data on performance of this exchange resin for secondary effluents, but little information on its long-term performance on septic tank effluents. Laboratory studies, detailed in Appendix A, indicated that clinoptilolite will effectively remove ammonia through at least 2 cycles with little loss in exchange capacity. Total cation capacity of 20 mesh resin was approximately 0.5 meq/g. The ammonia exchange capacity at breakthrough was approximately 0.4 meq/g with ammonia concentration in the septic tank effluent ranging from 24 to 30 mgN/L. Currently this process does not appear to be cost-effective for total household wastes but may be applicable to grey water treatment.

Disinfection

Where disinfection of effluent is required, several alternative systems have proven to be effective. The use of dry feed chlorinators employing calcium hypochlorite tablets will produce low levels of indicator organisms (Table 22). Unfortunately, a major problem associated with the use of these systems has been the lack of control of the dose to the wastewater. Periodic high chlorine concentrations were found in the treated effluents on an extended field testing program (Appendix A). Methods to more effectively control hypochlorite feed are needed. In light of the toxicity of chlorine, consideration must also be given to dechlorination of effluent prior to final surface water discharge.

Initial studies with ultraviolet irradiation of sand filtered household effluents have proven to be most promising. Four months of operating data with a commercially available UV unit are presented in Table 23. Long-term tests are continuing with these units in several field installations. One major drawback to UV irradiation is the high initial capital investiment. As greater demand for this type of system increases, costs will likely decrease, however. Details of the field testing of the UV equipment are found in Appendix A.

Other alternative methods for disinfection include iodine, bromine, formaldehyde and ozone. Iodine may prove to be a very practical disinfectant for low flow applications. It is an excellent bactericide and virucide and dose not react with organics in wastewater to the extent that chlorine does. Feeding of bromine salts appears to be too complex for small flow application, and ozone treatment also involves relatively complex equipment. Little experience has yet been gained with formaldehyde feed equipment.

TABLE 22. BACTERIAL CONCENTRATIONS FOLLOWING SAND FILTRATION AND CHLORINATION

Parameter	Site E Oct. 1973 - May 1974			Site E Aug. 1974 - Mar. 1975			Site H Sept. 1973 - Feb. 1975		
	Septic Tank	Sand Filter	Chlorine Contact	Septic Tank	Sand Filter	Chlorine Contact	Aerobic Unit	Sand Filter	Chlorine Contact
Total Bacteria (log#/L)	9.8	8.7	8.5	9.3	7.8	6.0	8.5	7.8	6.6
Total Coliform (log#/L)	7.3	5.2	3.3	7.0	4.1	1.5	6.2	5.2	2.5
Fecal Coliform (log#/L)	6.7	4.7	2.8	6.8	3.8	1.3	5.3	4.3	1.9
Fecal Streptococci (log#/L)	4.5	3.3	2.1	4.5	2.8	1.3	4.4	3.7	1.9
Ps. aeruginosa (log#/L)	-	-	-	4.5	2.4	<1.3	4.3	3.4	1.7

SECTION 6

SOILS AS A MEDIA FOR WASTWATER TREATMENT AND DISPOSAL

WASTEWATER ABSORPTION CAPABILITIES OF SOILS

Proper performance of conventional on-site wastewater disposal systems depends upon the ability of the soil or soil material to absorb and purify the wastewater. Failure occurs if either of these functions are not performed. Both are directly related to the hydraulic conductivity characteristics of the soil, which are largely controlled by the pore geometry of the material. Detailed discussion of this topic is contained in Appendix B.

Liquid Movement Through Soils

Soil Porosity and Permeability--

Soil is a complex arrangement of solid particles and pores filled with ever-changing amounts of air and water. The size and shape of these pores is a function of the soil's texture (particle size distribution) and its structure (the arrangement of the solid particles). In sandy soils, the voids are simply packing pores that exist between the individual grains. When significant amounts of clay and organic matter are present, soil particles adhere together to form aggregates or peds. Planar voids form separating the peds. Tubular channels formed by plants and animals living in the soil and irregularly shaped discontinuous pores, called vughs, are also found in soils (Figure 5).

Intrinsic soil permeability or the capability of the soil to conduct water is not determined by the soil porosity but rather the size, continuity and tortuosity of the pores. A clayey soil is more porous than a sandy soil, yet the sandy soil will conduct much more water, because it has larger, more continuous pores. These twisting pathways, with enlargements, constrictions and discontinuities through which the water moves, are constantly being altered as well. The soil structure, which helps to maintain the pores, is very dynamic and may change greatly from time to time in response to changes in natural conditions, biological activity and the soil-management practices. Repeated cycles of wetting and drying and freezing and thawing help to form peds, while plants with extensive root systems and soil fauna activity promote soil aggregation and channeling. On the other hand, mechanical compaction and the dilution of soluble salts can cause the breakdown of the peds, reducing the capacity of the soil to conduct water.

Characterization of Water in Soils--

Under naturally drained conditions, some pores in soil are filled with water. The distribution of this water depends upon the characteristics of the

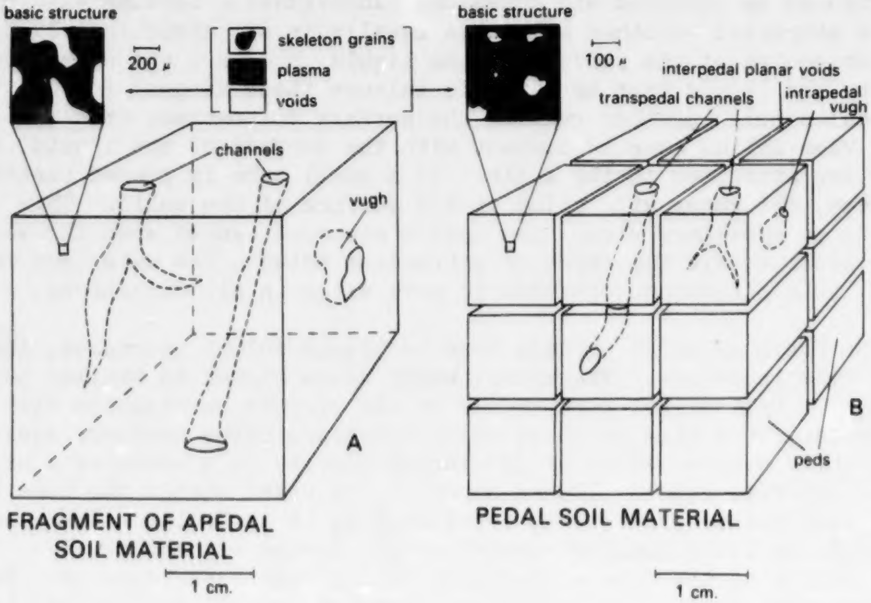

basic structure
200 µ
skeleton grains
plasma
voids
basic structure
100 µ
interpedal planar voids
transpedal channels
intrapedal vugh
channels
vugh
A
B
peds
FRAGMENT OF APEDAL SOIL MATERIAL
PEDAL SOIL MATERIAL
1 cm.
1 cm.

Figure 5. Schematic representation of a single-grained (left) and an aggregated soil material (right) (Bouma, et al. 1972).

pores while its movement is determined by the relative energy status of the water. Water flows downhill, but more accurately, it flows from points of higher potential energy to points of lower potential energy. The energy status is referred to as the moisture potential.

The total moisture potential Ψ_T in soils may be defined as:

$$\Psi_T = \Psi_m + \Psi_g + \Psi_p + \Psi_\pi + \Psi_\Omega$$

where Ψ_m, Ψ_g, Ψ_p, Ψ_π and Ψ_Ω are the matric, gravity, pressure, osmotic and overburden potentials, respectively. Of these, the matric, gravity and pressure potentials are the most significant in soil absorption of wastewater.

The gravitational potential Ψ_g is the result of the attraction of water toward the center of the earth by a gravitational force and is equal to the weight of the water. To raise water against gravity, work must be done and this work is stored by the water in the form of gravitational potential energy. The potential energy of the water at any point is determined by the elevation of that point relative to some reference level. Thus, the higher the water, the greater its gravitational potential.

The matric potential, Ψ_m, is produced by the affinity of water to the soil particle surfaces. The pores and surfaces of soil particles hold water due to forces produced by adhesion and cohesion. Individual molecules within the liquid are attracted to other molecules equally in all directions by cohesive forces. Molecules at the surface of the liquid, however, are attracted more strongly by the liquid than by air. To balance these unequal forces, the surface molecules pull together causing the surface to contract creating surface tension. When solids come in contact with the surface of the liquid, the water molecules are attracted to the solid. If a small tube is placed vertically in the water, the water will climb up the surface of the solid. This is referred to as capillary rise. The upward movement ceases when the weight of the raise water equals the force of attraction between the water and the solid. Of course, this phenomenon operates to move water in all directions.

As the ratio of solid surface area to liquid volume increases, the capillary rise increases. Therefore, water rises higher in smaller pores. For example, a cylindrical pore radius of 100 microns corresponds with a relatively low capillary rise of 28 cm water (pressure below meniscus equals -28 cm water) while a pore radius of 30 microns results in a relatively high rise of 103 cm (pressure equals -103 cm water). The water within the tube is at less than atmospheric pressure as noted because it is "pulled" downward by gravity as it is being "pulled" upward by the forces of capillarity. The water is under tension as the tube essentially "sucks" the water into it. This negative pressure in soil is called soil tension or soil suction and is measured in millibars (mbar). This implies that it takes more energy to remove or pull water from a small pore than a large one (Figure 6).

In addition to the capillary phenomenon, adhesive forces also contribute to the matric potential. Molecular forces between the surface of the soil particles and the water form envelopes of water over the particle surfaces retaining the water in the soil (Figure 6).

When the soil is saturated, all the pores in the soil are filled with water and no capillary suction occurs. The soil moisture tension or matric potential, therefore, is zero. If the soil drains, the largest pores empty first, because they have the least tension to hold water. As drainage continues, progressively smaller pores empty and the soil moisture tension increases because smaller pores have a stronger pull to hold water. Thus, the tension represents the energy state of the largest water filled pores. With further drainage, only the very narrowest pores and solid surfaces are able to exert sufficient "pull" to retain water. Hence, increasing tension or suction is associated with drying.

The rate of decrease of moisture in soil upon increasing tension is a function of its pore-size distribution, and is characteristic for each soil material or type. Figure 7 shows the soil moisture retention curves for a sand, silt loam, sandy loam and a clay soil. The sand has many relatively large pores that drain abruptly at relatively low tensions, whereas the clay releases only a small volume of water over a wide tension range because most of it is strongly retained in very fine pores. The silt loam has more coarse pores than does the clay, so its curve lies somewhat below that of the clay at higher tensions. The sandy loam has more fine pores and fewer coarse pores than the sand so its curve lies above that of the sand after initial drainage has occurred.

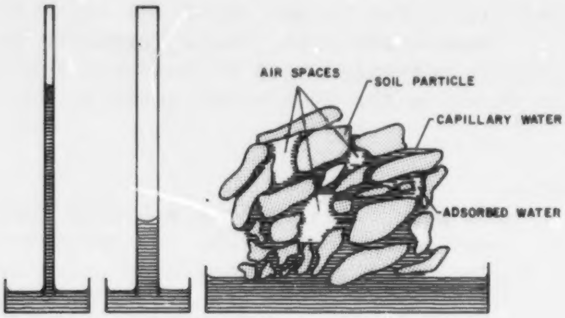

AIR SPACES
SOIL PARTICLE
CAPILLARY WATER
ADSORBED WATER

Figure 6. Upward movement by capillarity in glass tubes as compared with soils (after Brady, 1974).

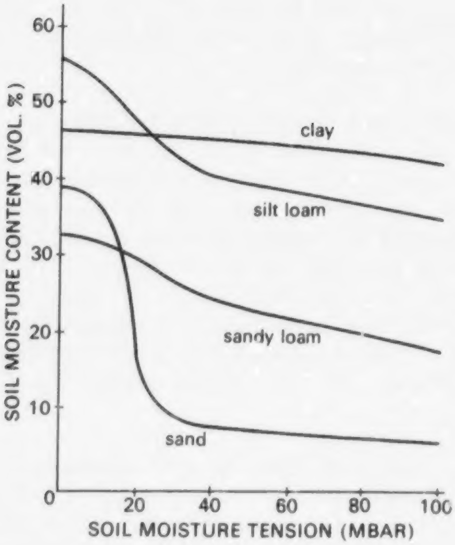


Figure 7. Soil moisture retention for four different soil materials (Bouma, et al. 1972).

The pressure potential is due to the weight of water at a particular point. If the point is beneath the water table, pressure potential is equal and opposite to the gravity potential that is measured from the free water surface. If the point is above the free water surface, the pressure potential is zero.

Liquid Movement in Soils--

Water will flow from a point where it has a higher energy potential to a point where it has a lower energy potential. In unsaturated soils, the gravity and matric potentials are the only significant components of the total energy potential since the pressure potential is zero. The gravity potential acts to move water vertically downward, while the matric potential acts to move water in all directions. In saturated soils, the pressure potential is analogous to the matric potential, now zero, in unsaturated soils. The rate of flow increases as the potential difference or potential gradient between points increases. The ratio of the flow rate to the potential gradient is referred to as the hydraulic conductivity or K defined by Darcy's Law.

$$Q = KA \frac{d\Psi}{dZ}$$

where: Q = flow rate
K = hydraulic conductivity
A = cross-sectional area of flow
$\frac{d\Psi}{dZ}$ = hydraulic gradient

The hydraulic conductivity, K, accounts for all the factors affecting flow within the soil including tortuosity and size of the pores. Thus, the measured K values for different soils vary widely due to differences in pore size distributions and pore continuity.

The hydraulic conductivity often changes dramatically with changes in the soil moisture tension. At a tension equal to or less than zero, the soil is saturated and all the pores in the soil are conducting liquid. When the tension is greater than zero, air is present in some of the pores and unsaturated conditions prevail. This condition grossly alters the flow channel or cross-sectional area, A, because the forces which cause flow are now associated with capillarity. As the water content decreases or tension increases, the path of the water flow becomes more and more tortuous since the water travels along surfaces and through sufficiently small pores to retain water at the prevailing moisture tension. Therefore, the unsaturated hydraulic conductivity is usually much lower.

To illustrate this, three different soil materials can be considered with pore size distributions schematically represented in Figure 8. One "soil" is a coarse, porous material (like a sand), one is a fine porous material (like a clay) and a third (like a sandy loam) has both large and fine pores. With an open infiltrative surface and with a sufficient supply of water, all the soil pores are filled and each pore will conduct water downward due to gravity. The large pores will conduct much more water than the smaller ones. If a weak

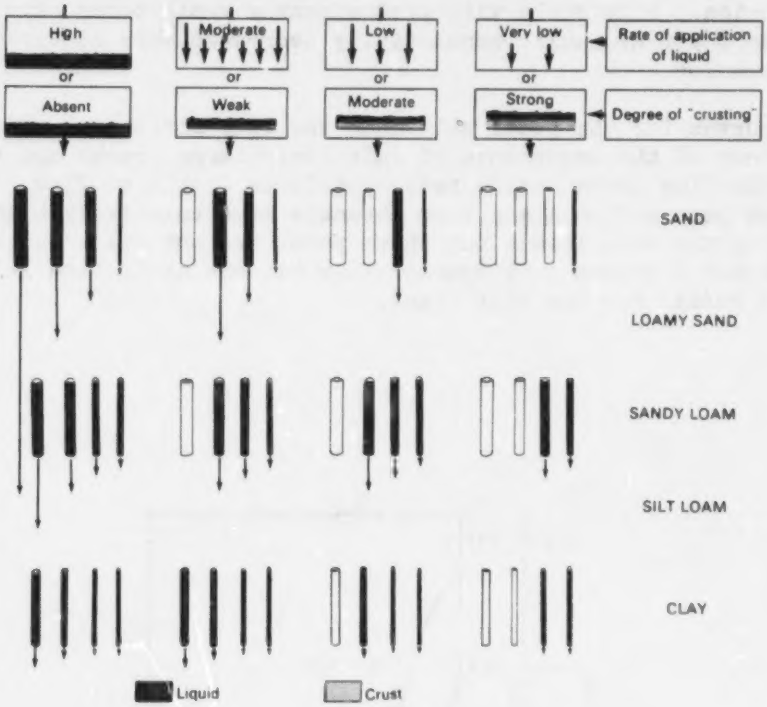

High
Moderate
Low
Very low
Rate of application of liquid
or
or
or
or
Absent
Weak
Moderate
Strong
Degree of "crusting"
SAND
LOAMY SAND
SANDY LOAM
SILT LOAM
CLAY
Liquid
Crust

Figure 8. Schematic illustration of the effect of increasing crust resistance or decreasing rate of application of liquid on the rate of percolation through three "soil materials" (Bouma et al., 1972).

barrier or crust forms over the tops of the tubes to restrict flow, some of the larger tubes will drain. Only the pores with sufficient capillary force to "pull" the water through the crust will conduct water. The larger the pore, the smaller the capillary force so that progressively smaller pores empty at increasing crust resistance. This crusting leads to a dramatic reduction in the hydraulic conductivity of the soil (Figure 8).

If no crust is present, similar phenomena occur when the rate of application of water to the soil is reduced. With abundant water supply, all pores are filled. If the supply is decreased, there is not enough water to keep all pores filled during the downard movement of the water. The larger pores empty first, since the smaller pores have a greater capillary attraction for water. Thus, larger pores can fill with water only if smaller pores have an insufficient capacity to conduct away all the applied water.

The reduction in K upon increasing soil moisture tension, therefore, is characteristic for a given soil texture and structure. Hydraulic conductivity or K curves, determined in situ show such patterns for natural soil (Figure 9). Coarse soils with predominantly large pores have relatively high saturated hydraulic conductivities (K_{sat}), but K drops rapidly with increasing soil moisture tension. Fine soils with predominantly small pores have relatively low K_{sat}, but their hydraulic conductivity decreases more slowly upon increasing tension.

The K curves for the pedal silt loam and clay horizons demonstrate the physical effect of the occurrence of relatively large cracks and root and worm channels. The fine pores inside peds contribute little to flow. The large pores between peds and root and worm channels give relatively high K_{sat} values (25 cm/day for the silt loam), but these pores are not filled with water at low tensions and K values drop dramatically between saturation and 20 cm tension (1.5 cm/day for the silt loam).

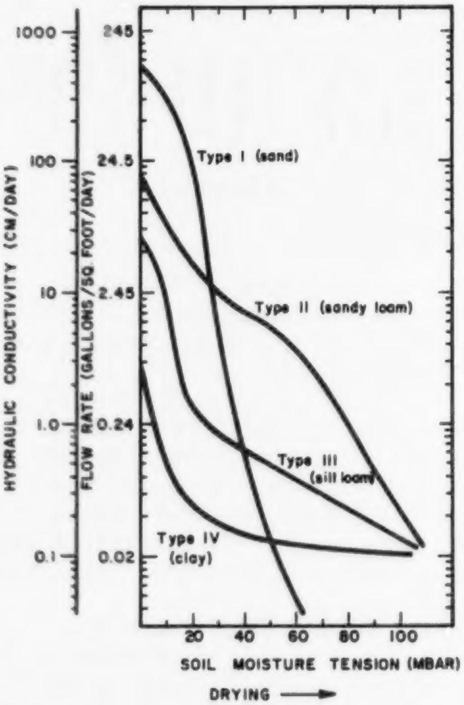


Figure 9. Hydraulic conductivity (K) as a function of soil moisture tension measured in situ with the crust-test procedure (Bouma, 1975).

Liquid Movement Into Soils

When liquid wastes are applied to the soil, a clogging zone often develops at the infiltrative surface. This restricts the rate of infiltration, preventing saturation of the underlying soil even though liquid is ponded above. The soil is then able to conduct liquid only if the water is able to penetrate the clogged zone under the forces of hydrostatic pressure and capillary pull.

The Process of Pore Clogging--

Several phenomena contribute to the development of a clogging zone at the infiltrative surface of soil absorption systems. These include: 1) puddling caused by the constant soaking of the soil during operation, 2) blockage of soil pores by solids filtered from the waste effluent, 3) accumulation of biomass from growth of microorganisms, 4) excretion of slimy polysaccharide gums by some soil bacteria, 5) deterioration of soil structure caused by exchange of ions on clay particles, and 6) precipitation of insoluble metal sulfides under anaerobic conditions.

This description of soil clogging assumes that the native soil structure is left relatively intact at the infiltrative surface during construction of the system. However, many systems fail, usually within a year or two, because of poor construction techniques. Absorption of water by soils depends upon preservation of a suitable soil structure, but soil structure can be partially or completely destroyed by compaction and smearing during construction. Extensive damage does not occur in soils with a single-grained structure (sands), but can be very serious in aggregated soils with high clay contents. When mechanical forces are applied to a moist or wet soil, the water around clay particles acts as a "lubricant" causing the soil to exhibit plasticity where individual particles move relative to one another. Such movements, referred to as compaction, puddling, or smearing, close the larger pores. The potential for structural damage of this type increases as soil wetness and clay content increase. Compaction may result from frequent passes over the field by heavy machinery, smearing of the soil surface by excavating equipment and puddling by exposure of the infiltrative surface for a day or more to rainfall that seals off the soil pores. The result is that the absorption field may be clogged before it is put into service.

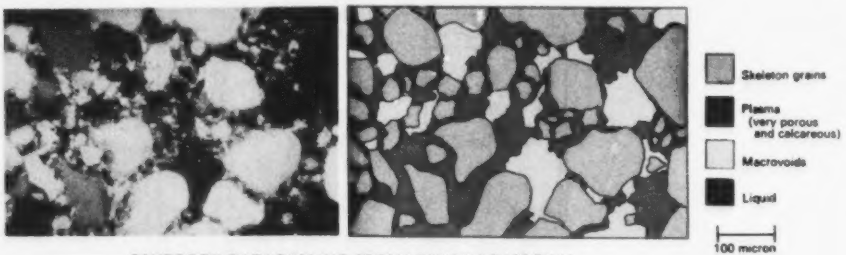
Studies by several investigators indicate that the physical and biological mechanisms are the primary causes of soil clogging in an absorption field not smeared and compacted during construction (Bendixen, et al. 1950; Bouma, et al. 1972; de Vries, 1972; Laak, 1970; McGauhey and Krone, 1967; McGauhey and Winneberger, 1964; Weibel, et al. 1954). In these instances, clogging seems to develop in three stages: 1) slow initial clogging, 2) rapid increase of resistance, leading to permanent ponding, and 3) a final leveling off towards equilibrium. During initial development of the clogging zone, the liquid seeps away more and more slowly between loadings. Aerobic bacteria decompose many of the organic solids, helping to keep the soil pores open, but they can function only when the infiltrative surface drains between doses to allow the entry of air. As the clogging zone begins to form, decreasing the aerobic periods between ponding, the aerobic bacteria eventually are unable to keep up with the influx of solids. Permanent ponding finally results, leading to anaerobic conditions where oxygen is no longer present. Any dissolved oxygen in the water is inadequate to maintain the aerobic environment necessary for the rapid decomposition of the organic matter. Clogging then proceeds more quickly due to the less efficient destruction of soil clogging organics by anaerobic bacteria. Sulfides produced by reduction of sulfate by these bacteria bind up trace elements as insoluble sulfides, causing heavy black deposits in the clogging zone. Some anaerobic and facultative organisms, which grow in such an environment, produce gelatinous materials (bacterial polysaccharides, slimes, or gums), which clog the soil pores very effectively. At this point, the clogging mat seems to reach an equilibrium state where the resistance to flow changes little.

The Significance of Soil Clogging--

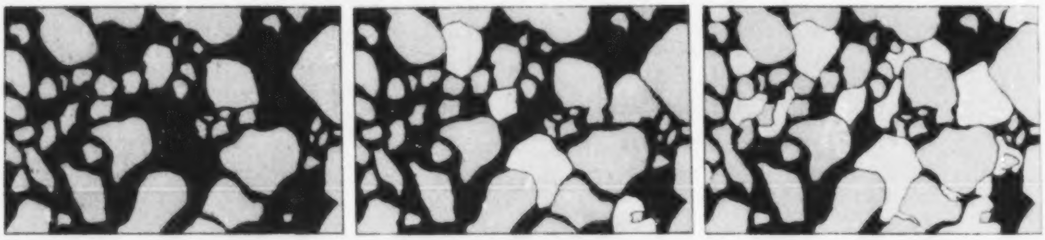
Because of the barrier to flow created by soil clogging, the soil below the clogged area becomes unsaturated. This is very significant when wastewater is applied to the soil for disposal. Flow of liquid in unsaturated soil proceeds at a much slower rate than in saturated soil, because flow only occurs in the finer pores. This slows the rate of infiltration into the soil, but enhances purification. Wastewater effluent is purified by filtration, biochemical reactions and adsorption processess which are more effective in unsaturated soils because average distances between effluent particles and the soil particles decrease, while the time of contact increases. This flow phenomenon can be illustrated by an example (Bouma, et al. 1972). Figure 10 shows a thin section of the C horizon of a Saybrook silt loam, which is a stony sand loam till with a saturated hydraulic conductivity of 80 cm/day. The flow velocity of water in the soil pores can be estimated knowing the percent of water filled pores at different moisture potentials as given by its moisture retention curve (Figure 7). This velocity can be used to derive the time for water to travel one foot (30 cm), assuming a hydraulic gradient of 1 cm/cm due only to gravity. Successively smaller pores empty at increasing tensions and K decreases correspondingly (Figure 9). Calculated travel times increase from 3 hours at saturation to 30 hours at 30 mbars and 8 days at 80 mbars of soil moisture tension.

In structured soils it is possible to have flow predominantly through the planar voids, thus bypassing the interior of the peds. High liquid applications may result in high dispersion where the water passes through the planar voids without displacing the water already in the peds. In such instances, short circuiting of liquid through the soil occurs with associated lower retention times. Lower application rates would displace more of the water in the peds and reduce dispersion. Differences in dispersion related to different soil structures have been noted in lysimeter studies of chloride movement in soil. The soil columns indicated a short circuiting to be a particular problem on drained soils dosed at relatively high rates (Anderson and Bouma, 1977a and 1977b).

Short circuiting in a structured soil is schematically illustrated in Figure 11. If the large planar voids are drained and air filled, a high application rate of liquid applied at the surface will quickly pass through the large pores before much can enter the fine pores of the peds. Therefore,



SAYBROOK SILT LOAM (IIC STONY SANDY LOAM TILL)



SATURATED
K = 80 cm/day
ONE FOOT (30 cm) MOVEMENT IN THE SOIL IN : 3 hours
(hydraulic gradient : 1 cm/cm)

At 30 mb. SUCTION
K = 7 cm/day
30 hours

At 80 mb. SUCTION
K = 7 mm/day
8 days

Figure 10. Occurrences and movement of liquid in a saturated and unsaturated sandy loam till C horizon of Saybrook silt loam (Bouma, et al. 1972).

channeling occurs where the retention time of the bulk of the liquid is low and only a portion of the entire soil volume is used to transmit the fluid. If the application rate is low or if there is a barrier to flow, such that the soil remains unsaturated, the dispersion is low. The large pores will not fill with liquid and flow will be through the finer pores within the peds. In this case, the retention time will be longer and flow will be primarily through the portion of the soil most effective in renovation.

Long liquid travel times are desirable to adequately purify the wastewater. The design of absorption systems may be critical to achieve this in some soils. Travel times are sufficiently long under all moisture tensions to effect adequate purification in clay, but are too short in sand and sandy loams when the soil is near saturation. Once a clogging zone has developed in such permeable soils, the hydraulic conductivity is reduced to a level where sufficiently long travel times result. However, when an absorption system constructed in a highly porous or dry structured soil is first put into service without a developed clogged zone, adequate purification may require an increased depth of soil unless precautions are taken in design to insure unsaturated soil conditions are maintained.

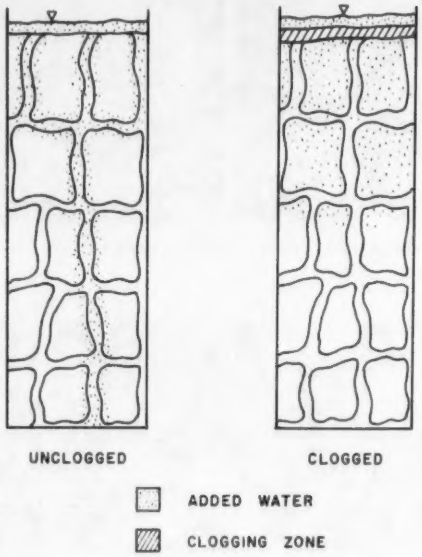


Figure 11. Influence of clogging zone on short circuiting in structured soils.

Factors Effecting Soil Clogging--

Dosing and resting--There is substantial evidence that continuous ponding of wastewater on the soil's infiltrative surface leads to more severe clogging than if the clogging mat were to remain at least intermittently aerobic (Bendixen, et al. 1960; Winneberger, et al. 1960; and Thomas, et al. 1966). To provide reaeration, periods of loading are followed by periods of resting with cycle frequencies ranging from hours to months. The resting phase allows the soil to drain and reaerate, thus encouraging rapid degradation of the clogging mat. This operation may extend the life of an absorption system or reduce the infiltrative surface area by keeping the clogging mat resistance to a minimum.

Early laboratory work with lysimeters showed repeatedly that reduction in the infiltrative capacity of the soil proceeds more slowly when periods of ponding were alternated with periods of aeration (Bendix, et al. 1950; Winneberger, et al. 1960; and Thomas, et al. 1966). Contrary to these findings, Kropft, et al. (1975; 1977) report that total flow through the clogging mat remained higher in constantly ponded soil columns than in columns subjected to intermittent flooding. Similar results were obtained in this study when comparisons were made between soil columns aerated below the infiltrative surface and those that were not (Perry and Harris, 1975; Jawson, 1976). The aerated columns showed that effluent application regimes characterized by short term alternating anaerobic-aerobic conditions may result in reduced infiltration associated with the formation of an intense clogging during the aerobic resting phase. Once clogged, restoration of the infiltrative surface by resting requires at least three to four weeks in sands (Perry and Harris, 1975). The required resting period may be longer in finer textured soils.

These results may not be as contradictory as they first seem. The oxidation-reduction potential in and around the clogging mat may be critical to maintaining high infiltration rates. Initially, cycles of dosing and resting maintain higher redox potentials in the soil than continuous ponding, which retards the development of the clogging mat. However, if clogging is allowed to proceed, the organics accumulated during periods of dosing may be too great for complete aerobic digestion during the resting phase. With an ample food supply, the aerobic and facultative organisms rapidly convert the clogging agents to new cell mass and slime which become new clogging agents. To prevent this, longer periods of aeration or more uniform distribution may be necessary to realize any benefits of dosing.

The laboratory results have not yet been validated in the field. Limited data from existing dosing systems indicate that the mechanisms may be even more complex than indicated. Bouma, et al. (1975a) and the University of Wisconsin reported that a system constructed in a silty clay loam soil with a percolation rate of 12 min/cm (30 min/inch) was still operating satisfactorily when dosed once daily at a loading rate one-third that recommended by the Manual of Septic Tank Practice (U.S. Public Health Service, 1967). When excavations were made to determine the extent of clogging, evidence of worm activity was observed in the clogging mat which seemed to reopen the infiltrative surface. This activity could only occur during periods when the soil is not ponded. More field work is necessary to determine optimum cycles of dosing and resting.

Applied wastewater quality--It is reasonable to assume that improving the wastewater quality before application to the soil would inhibit clogging. However, in several column studies investigating the effects of effluent clogging on soil infiltration, only slight differences in clogging rates were found over a broad range of wastewater qualities.

In studies with undisturbed cores of Almena silt loam (percolation rates of 70 min/inch in the topsoil and 100 min/inch in the subsoil) columns were continuously ponded with septic tank effluent, aerobic unit effluent and distilled water (Daniel and Bouma, 1974). The aerobic effluent had a significantly lower biodegradable organic concentration than the septic tank effluent (chemical oxygen demand concentrations of 60 mg/L and 150 mg/L respectively, but the suspended solids concentrations were similar (33 mg/L and 40 mg/L respectively). More severe clogging occurred with the aerobic effluent. No clogging occurred in the soil loaded with distilled water. It was hypothesized that finely divided suspended solids in the aerobically treated wastewater were able to enter the small pores in the soil, clogging the soil with depth and creating a more effective barrier to flow.

Subsequent studies designed to test the solids clogging hypothesis, indicated that the initial saturated hydraulic conductivity is more significant than effluent quality. Undisturbed cores of Almena silt loam were paired according to their initial saturated hydraulic conductivity, one pair representing a high and low initial K_{sat}. Three sets of four replicates each were dosed with 1 cm/day of septic tank effluent (48 mg/L BOD_5, 28 mg/L TSS), aerobic unit effluent (27 mg/L BOD_5, 61 mg/L TSS), and tap water. The length of time to when each column remained ponded between daily doses was recorded.

The aerobic columns showed mean ponding times of 21.3 weeks, the septic tank set 20.6 weeks, and the tap water 18.3 weeks. When initial K_{sat} values were compared between all three sets, the ponding times for the high K_{sat} columns was 28 weeks while the ponding times for the low K_{sat} was 14.8 weeks.

These studies seem to indicate that, in fine texture structured soils, applied effluent quality does not affect the rate of clogging. The sand filter studies, however, suggest that improved quality may reduce the degree of clogging in coarse granular soils.

Infiltration Rates Through Clogged Surfaces--

If the soil's ability to accept liquid during wastewater application is to be accurately predicted, consideration of unsaturated flow phenomena due to soil clogging mats or compaction is essential. Clogging mats or compacted soil layers of progressively higher resistances will allow progressively lower rates of infiltration through the soil.

Infiltration is not only dependent upon the resistance of the clogging zone, but also on the hydrostatic pressure of the ponded water above the clogging layer and the capillary properties of the underlying soil (Bouma, 1975). For example, an identical "crust" with a resistance of 5 days (the length of time for 1 cm^3 to pass through 1 cm^2 of barrier with a head of 1 cm) ponded with 5 cm of liquid, would induce flow rates of 8 cm/day in a sandy loam; 7 cm/day in a sand; 4 cm/day in a silt loam; and 1.8 cm/day in a clay. "Crusts" with very high resistances would conduct more liquid when overlying a clay than when overlying a sand. Thus, if similar clogging zones developed in different soils, they would have different conductivities. This fact may effect clogging mat development and its ultimate resistance.

The hydraulic conductivity, which is the one-dimensional flow rate through a unit area under a unit hydraulic gradient, is a reliable measure or any saturated or unsaturated soil to accept and conduct liquid. Figure 9 presents the general K-curves developed for the major textural groupings in Wisconsin. These curves relate K to the soil moisture tension. Continued research may result in different groups at a later date. Through the use of tensiometry, the soil moisture potential and gradient around soil absorption beds can be measured in situ with little disturbance of the soil. Thus, the moisture potential and its gradient measured in situ can be translated into a flow rate by using Darcy's Law (Bouma, 1975).

Moisture potentials were measured under several ponded conventional septic tank-soil absorption systems to determine equilibrium flow rates through clogging mats in different soils (Bouma, 1975; Bouma, et al. 1972; 1975a; Magdoff and Bouma, 1974; Walker, et al. 1973a). Small excavations were made adjacent to ponded systems and the tensiometers were installed at different points in the soil below and to the side of the systems. Measured potentials were used to estimate infiltration rates into the soil through the bottom and the sidewalls of the system, using the appropriate K-curve (Figure 9). A summary of the results is presented in Table 23.

Conductivity type I (sands)--Results of monitoring systems installed in sands showed that moisture tensions and associated flow rates in soil surrounding clogged trenches or beds were not very different for different systems, despite differences in system age. This would indicate that a mature clogging mat is established early in the system's life and that flow rates through the mat change little as the system ages. The results also show that clogged sands accept significant quantities of septic tank effluent through both bottom and sidewall surfaces.

Based on these data, 5 cm/day (1.2 gpd/ft^2) is recommended as a maximum loading rate of the bottom area for systems with 30-cm (12 in) sidewalls constructed in sands (Bouma, 1975). It is further recommended that the effluent be applied uniformly over the entire bottom infiltrative surface in four or more daily doses, particularly during system start-up, if bacterial and viral contamination of a shallow water table is a concern. The uniform application in small volumes will insure unsaturated conditions in the sand necessary for good purification.

Conductivity type II (sandy loams, loams)--Soils of this type have rapid percolation rates initially, but they have a tendency to clog quite severely. This may be due to their particular pore size distribution and structural instability. Relatively low clay contents do not allow significant swelling and shrinking of the soil necessary to form structural units or peds with associated interpedal cracks. Tubular worm and root channels are formed, but they tend to be more unstable and much less permanent than those formed in clayey soils. Thus, the packing pores between particles, which are much finer than in the sands, are the principal voids through which the water moves (Bouma and Anderson, 1973). The finer pores may result in greater accumulation of solids at the infiltrative surface and the development of anaerobic conditions in the clogged layer due to the reduced air diffusion compared to sandy soils (Magdoff, et al. 1974a).

The moisture tensions measured below the bottom of the systems increased with increased ponding depth within the systems. The estimated rates through the bottom areas varied from 0.4 cm/day (0.09 gpd/ft^2) to 1.9 cm/day (0.45 gpd/ft^2). The estimated rates through the sidewalls were similar.

To maintain reasonable infiltration rates, the data suggest that ponding levels within the system should be kept to a minimum. To reduce ponding levels, intermittent periods of aeration between applications should be provided to allow aerobic decomposition of the clogging mat. To test this hypothesis, one system was drained and allowed to dry before wastewater was reapplied in once per day dosings. After several months of operation in this mode, the moisture tensions below the clogging mat had dropped from 80 mbar to 60 mbar, indicating the mat was passing more liquid. When the operation returned to continuous application, the moisture tensions again increased to 80 mbar (Bouma, et al. 1972).

Absorption fields designed for bottom area loadings of 3 cm/day (0.7 gpd/ft^2) with 30-cm (12-in) sidewalls have functioned well in Wisconsin. Trenches are preferred to beds, with once daily dosing recommended if this rate is used (Bouma, 1975). This rate is somewhat lower than design rates used elsewhere.

Conductivity type III (silt loams, some silty clay loams)--Although these soils are more finely textured than either Type I or Type II soils, their more strongly structured nature can maintain relatively high infiltration rates if the system is constructed and managed properly. The cause of many failures can be traced to construction problems (Bouma, 1975; Bouma, et al. 1975a). Construction of beds often involves several passes over the infiltrative surface by machinery while excavating and placing of the rock. This practice can result in severe compaction and puddling if the soil is wet, because these finer textured soils exhibit a plastic consistency over a wide range of moisture contents, which occur naturally in the field (Bouma, 1975). Observations made at some installations indicated that excavating equipment had been driven over the bottom areas of the beds during construction. The presence of a compacted layer was confirmed by moisture tension measurements taken below the beds. These indicated a restricting layer with a resistance reasonbly close to resistances through layers of manually puddled fine silty soil materials (Bouma, et al. 1971; Bouma, et al. 1975a). Other systems studied, which were functioning satisfactorily, did not contain ponded effluent. Samples taken of the soil from the bottom of the systems showed well exposed soil structure with open planar voids between peds as well as worm and root channels. The exposure of these larger pores explains the lack of ponding. This points to the importance of construction practices which minimize damage to the structure of the soil.

Dosing and uniform distribution, with drying periods under aerobic conditions between applications, may stimulate worm and other fauna activity, as organisms seek the nutrient deposited at the infiltrative surface. Such activity can leave relatively large open channels through the clogging mat. This would seem to suggest that while good construction practices are necessary to expose an open infiltrative surface, periodic application of effluent is essential to keeping the surface open.

While the data are not conclusive, they suggest that maximum permissible loading rates would vary according to the method of distribution employed. If once daily dosing were employed, maximum rates of 5 cm/day (1.2 gpd/ft^2) might be acceptable based on bottom area only (Bouma, 1975). Uniform distribution would be crucial in this case to maintain unsaturated flow so that deep penetration of pollutants through the large exposed pores will not occur. If conventional gravity trickle distribution is used, the conventional loading rate of 2 cm/day (0.5 gpd/ft^2) should not be exceeded. In both cases, shallow trench designs 45 to 60 cm (18 to 24 in) deep are preferred because the upper soil horizons are usually more porous and less subject to damage during construction. Shallow systems also enhance evapotranspiration.

Conductivity type IV (clays, some silty clay loams)--Low conductivities in these soils at saturation drop strongly in the 0 to 20 mbar tension range due to the emptying of the interpedal voids and tubular channels as in Type III soils. However, lower K_{sat} values indicate the lack of many large pores. Thus, the soil itself, rather than the clogging mat, becomes the dominant controlling factor (Bouma, 1975; Healey and Laak, 1974).

Because soils of this type have severely limiting hydraulic properties, it may be more crucial to maintain an open infiltrative surface to utilize the large interpedal cracks and tubular channels. Dosing frequencies of once per day or longer may promote soil fauna activity between dosings to maintain an open surface (Bouma, et al. 1975a). If conventional gravity distribution is used, loading rates of 1 cm/day (0.2 gpd/ft^2) based on the bottom area only would seem to be acceptable, assuming 33 percent of the flow would be through the sidewall (Bouma, 1975). If expandable clays are present, a lower rate should be used.

TABLE 23. RECOMMENDED MAXIMUM LOADING RATES FOR SEPTIC TANK SOIL ABSORPTION FIELDS BASED ON IN SITU MEASUREMENTS[1] (After Bouma, 1975)

Conductivity Type	Soil Texture	Loading Rate[2] cm/day (gpd/ft^2)	Operating Conditions
I	Sand	5 (1.2)	4 doses/day Uniform Distribution Trenches or Beds
II	Sandy Loams	3 (0.7)	1 dose/day Uniform Distribution Trenches Preferred
	Loams	2 (0.5)	Conventional Distribution Shallow Trenches
III	Silt Loams Some Silty Clay Loams	5 (1.2)[3]	1 dose/day Uniform Distribution Shallow Trenches Only
IV	Clays	1 (0.2)[3]	1 dose/day Uniform Distribution Desirable Shallow Trenches Only

[1] Assumes that the high water table is > 90 cm (3 ft) below the infiltrative surface.

[2] Bottom area only

[3] Should not be applied to soils with expandable clays.

WASTEWATER TREATMENT CAPABILITIES OF SOILS

The principal goal in soil disposal of liquid wastes for homes in unsewered areas is the purification of the liquid before it reaches surface or ground waters. Organic matter, chemicals and pathogenic organisms and viruses that are not removed prior to application to the soil must be removed or transformed by the soil material. Numerous studies have shown that under proper conditions, the soil is an extremely efficient purifying medium. More detailed discussion on this topic can be found in Appendix C.

The Fate of Bacteria and Viruses in Soils

From the standpoint of public health, the removal of potential pathogenic organisms and viruses is the most critical function of a soil absorption system. Many field and laboratory studies have examined the efficiency of the soil for pathogen removal and the various parameters that affect its efficiency. Factors important in removal of pathogens by soil include soil type, temperature, pH, organism adsorption to soil and soil clogging materials, soil moisture and nutrient content and biological antagonisms (Gerba, et al. 1975). Another key factor is the liquid flow regime in the soil. As shown previously, unsaturated flow, induced by either a clogged zone or application rate, enhances purification because liquid movement is primarily through only the smaller pores of the soil forcing greater liquid-soil contact.

Figure 12 shows removal of fecal coliforms and fecal streptococci from septic tank effluent by two columns packed with 60 cm (2 ft) of Plainfield loamy sand (effective size 0.14 mm, uniformity coefficient 1.99) (McCoy and Ziebell, 1975; Ziebell, 1975). Both columns were loaded well below their saturated hydraulic conductivity rates of nearly 400 cm/day (96 gpd/ft^2), but one was loaded at twice the rate of the other. During the first 100 days of application, the number of bacteria discharged from both columns reached a plateau and then began to decline. Fewer bacteria passed through the column with the lower loading rate. Column 1, loaded at 10 cm/day (2.4 gpd/ft^2), removed approximately 92 percent of the fecal coliforms applied per day while column 2, loaded at 5 cm/day (1.2 gpd/ft^2), removed 99.9 percent. Fecal streptococci and Pseudomonas aeruginosa were also found in the effluent from the more heavily loaded column 1. These organisms were not detected in effluent from the more lightly loaded column 2. During this period a clogging zone developed on the infiltrative surface of each column and the fecal coliform count in the effluents from both columns eventually dropped to between 10 and 100 FC/100 mL (McCoy and Ziebell, 1975).

Septic tank systems installed in sands also exhibit the effects of the clogging zone in removing indicator bacteria. Figure 13 shows the bacterial counts obtained while monitoring several points around an absorption trench in an unsaturated medium sand soil. The kinds and numbers of bacteria found in the liquid 1 foot (30 cm) below and 1 foot (30 cm) to the side of the trench were similar to natural soil flora (Bouma, et al. 1972; McCoy and Ziebell, 1975; Ziebell, 1975).

Concurrent studies of Almena silt loam were also conducted (Ziebell, 1975). This soil has a lower capacity to conduct liquid than the unstructured sands and the majority of flow is through the larger pores between soil peds. Undisturbed cores, 60 cm (2 ft) deep, of Almena silt loam were loaded with septic tank effluent at a rate of 1 cm/day (0.24 gpd/ft^2). At this loading, with no clogging mat present, effluent short-circuited through large pores and channels in the soil and significant numbers of bacteria were found in the column effluents. When the loading rate was reduced to 0.3 cm/day to promote flow through the soil peds rather than through the larger cracks around the peds, counts decreased dramatically to below 2/100 mL of fecal coliforms,

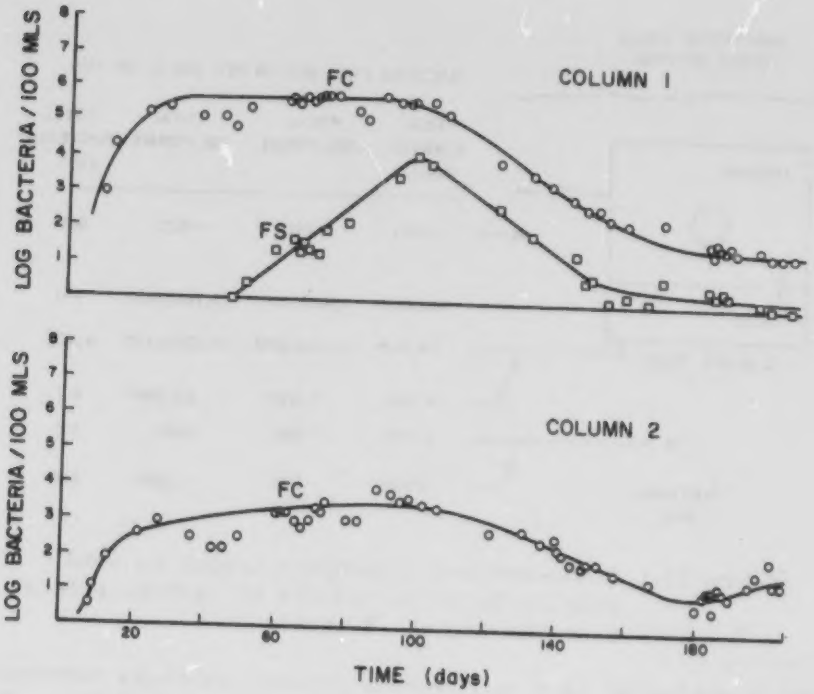
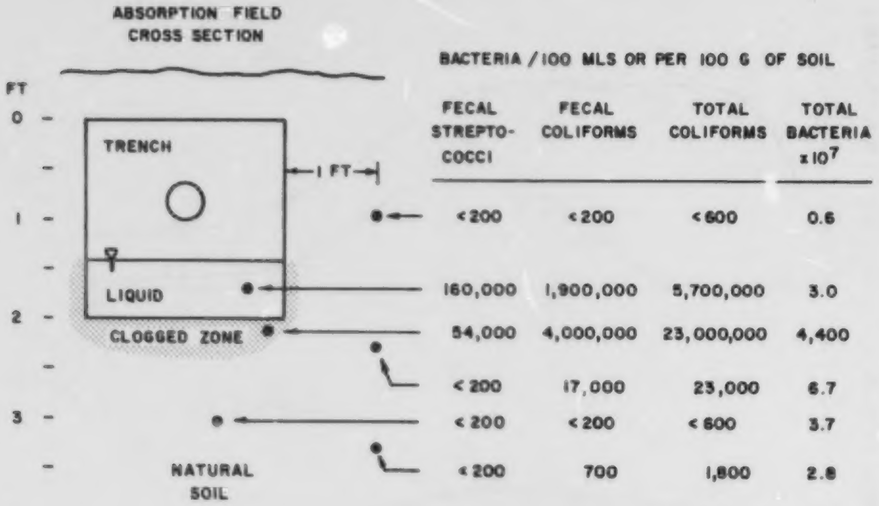


Figure 12. Bacteria counts in effluents from sand columns loaded with septic tank effluent. Column 1 loaded at 10 cm/day (1.5 hours retention time) and Column 2 loaded at 5 cm/day (25 hours retention time). FC = fecal coliforms; FS - fecal streptococcus (Ziebell, 1975).

fecal streptococcus and P. aeruginosa (Figure 14). When the loading was restored to 1 cm/day, high counts of these organisms were again observed.

Virus adsorption and inactivation in soils have been of considerable interest to scientists and engineers over the years. When viruses enter the septic tank or other treatment process, they are likely associated with cells in fecal material. These solids settle, but they may release some viruses depending upon turbulence within the process. Secondary adsorption on wastewater solids may occur in treatment processes but some free and particle adsorbed viruses will be discharged to subsequent treatment processes or to the soil absorption field.

Removal of viruses in soils occurs as the result of the combined effects of sorption, inactivation and retention. Upon entry into the soil, viruses are rapidly adsorbed to solid surfaces. Desorption appears to be strongly related to the ionic strength of the applied fluid, increasing as the ionic strength decreases (Lance, et al. 1976). In the adsorbed position, the


ABSORPTION FIELD CROSS SECTION
FT
0
1
2
3
TRENCH
1 FT
LIQUID
CLOGGED ZONE
NATURAL SOIL
BACTERIA / 100 MLS OR PER 100 G OF SOIL

FECAL STREPTO-COCCI	FECAL COLIFORMS	TOTAL COLIFORMS	TOTAL BACTERIA x10^7
<200	<200	<600	0.6
160,000	1,900,000	5,700,000	3.0
54,000	4,000,000	23,000,000	4,400
< 200	17,000	23,000	6.7
< 200	< 200	< 600	3.7
< 200	700	1,800	2.8

Figure 13. Cross-section of seepage trench in sand showing bacterial counts at various points near the trench (Ziebell, 1975).

the viruses are inactivated in a spontaneous process which is temperature dependent, being greater at the higher temperatures (Green, 1976). Virus detention within the soil is affected by the degree of saturation of the pores through which the virus laden effluent flows. The more saturated the pores, the less opportunity there is for virus contact with surfaces to which it can adsorb.

In laboratory studies with packed sand columns, septic tank effluent was inoculated with more than 10^5 plaque-forming units (PFU) per liter of poliovirus type 1 (Green, 1976; Green and Cliver, 1974). All viruses were removed in the columns at a loading rate of 5 cm/day (1.24 gpd/ft^2) applied in single doses over a period of more than one year. At a loading rate of 50 cm/day (12.4 gpd/ft^2), virus breakthrough occurred (Figure 15). Analysis of the sand residue following virus application, indicated that adsorbed viruses within the column were inactivated at a rate of 18 percent per day at room temperature and at 1.1 percent per day at 6 to 8°C (Green and Cliver, 1974).

Laboratory tests with ground soil material from a Batavia silt loam reduced virus in septic tank effluents by 5.4 logs per cm of depth and Almena silt loams material produced 7.9 logs of reduction per cm (Green and Cliver, 1974). It should be emphasized, however, that soils in the field do not exist in a finely ground state. Channels in natural soil will reduce opportunity for virus adsorption and travel over long distances may occur when loading rates are high.

It would appear from these studies that sandy soils without structure, loaded at 5 cm/day (1.24 gpd/ft^2) or less, should adequately remove bacteria

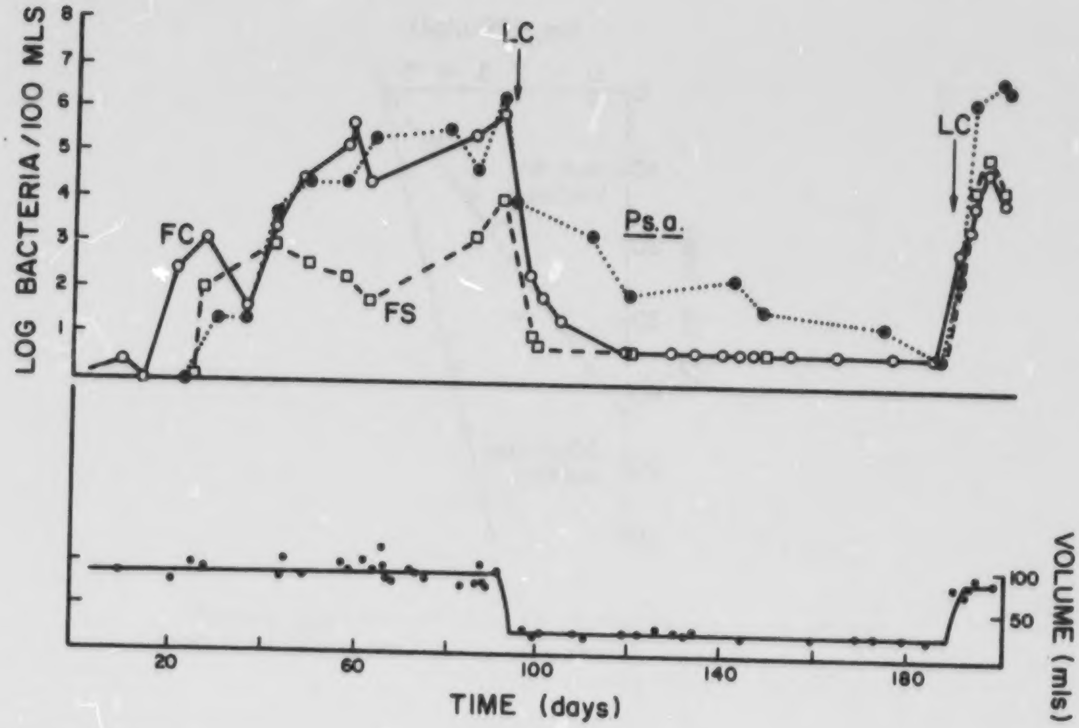


Figure 14. Bacteria counts in effluent from an undisturbed core of Almena silt loam loaded with septic tank effluent (Ziebell, 1975).

and viruses within 60 cm (2 ft). In structured soils with no clogging mat present, lower loading rates are required to achieve purification within 60 cm (2 ft). In clogged soils where infiltration rates are limited by the clogging mat, loading rates are less critical unless surfacing of effluent occurs due to excessive applications.

The Fate of Nutrients in Soils

Domestic wastewaters may contain chemicals hazardous to public health or the environment. Nitrogen and phosphorus compounds are discharged in household wastewater which can enter ground or surface waters in sufficient quantities to cause concern. Nitrogen, in the form of nitrate or nitrite has been linked to cases of methemoglobinemia in infants (Groundwater Contamination, 1961). A safety limit for nitrate of 10 mg/L as nitrogen is recommended by the U.S. Public Health Service (Gruener and Shuval, 1969). There are many reports of nitrate concentrations above 10 mg/L-N limit in wells near septic tank systems (Dudley and Stephenson, 1973; Preul, 1964; Walker, et al. 1973b; Groundwater Contamination, 1961).

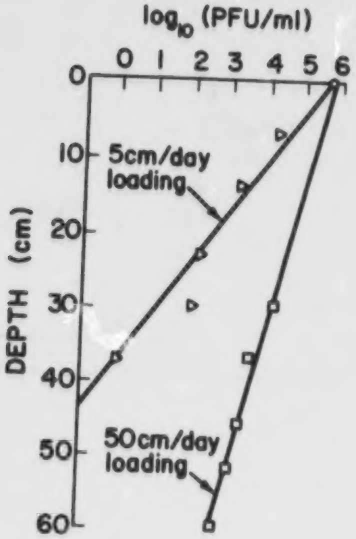


Figure 15. Penetration of poliovirus into packed sand columns at room temperature (Green and Cliver, 1974).

In solution, nitrate moves freely through the soil. Some denitrification (reduction of nitrate to nitrogen gas) can occur where organic material and an anaerobic environment occur together. Nitrogen in septic tank effluent is about 80 percent ammonium and 20 percent organic nitrogen, but much of it is converted biologically to nitrate as it moves through the aerated unsaturated soil immediately below the clogging zone in the seepage field (Walker, et al. 1973b). This is illustrated in Figure 16 where concentrations of the various forms of nitrogen are plotted against depth below a soil trench in a sandy soil. If anaerobic conditions prevail in the subsoil, nitrification will not occur and the nitrogen then remains in the form of ammonium. Ammonium is readily adsorbed by soil materials of high clay content and hence migrates much more slowly (Preul, 1964; Walker, et al. 1973b).

In absorption fields where several feet of unsaturated flow in aerobic soil occurs, nitrification followed by leaching of the nitrate into the groundwater results. Some denitrification may occur in anaerobic "micro-environments" within peds, but dilution is the primary mechanism available to reduce nitrate concentrations to safe levels. In conditions of high groundwater or very slowly permeable soils, anaerobic soil conditions may exist. Under these conditions, nitrification is avoided and adsorption of ammonia onto the clay and organic fraction of the soils occurs (Bouma, et al. 1972; Dudely and Stephenson, 1973; Preul, 1966). As adsorption sites become exhausted, ammonium travels. Most of the ammonium is subject to nitrification and leaching if aerobic conditions are reestablished (Lance, 1972).

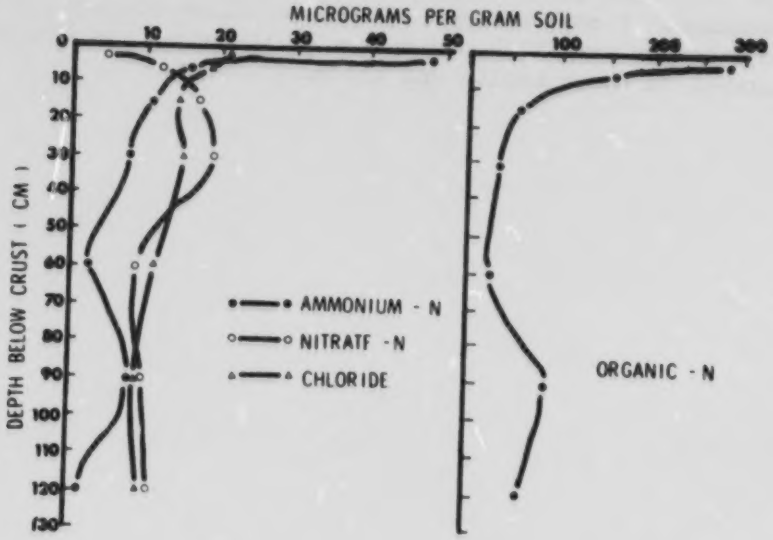


Figure 16. Concentrations of NH_4-N, NO_3-N, Organic N and Cl in unsaturated soil below the clogged zone in sand (Walker, et al. 1973b).

Phosphorus is also of environmental concern. If allowed to reach surface waters, it can accelerate eutrophication because it is an essential nutrient of algae and aquatic weeds. However, phosphorus enrichment of groundwater seldom occurs below septic tank systems because phosphorus is fixed in soil by sorption reactions or as phosphate precipitates of calcium, aluminum or iron.

When phosphorus is initially applied to the soil, it can be chemisorbed on the soil mineral surfaces. As the concentration of phosphorus increases in the soil solution, phosphate precipitates may form. The phosphate ion forms relatively stable surface compounds or precipitates with compounds containing iron or aluminum in neutral to acid soils or calcium in neutral to alkaline soils.

The amount of phosphorus retained by a soil can be significant. In a sandy loam soil, 100 mg/g to 300 mg/g of phosphorus was retained below a soil absorption system (Walker, et al. 1973b). In sand column studies, 121 mg/g of phosphorus was retained (Magdoff and Keeney, 1976). Based on this and data collected by other investigators, it is estimated that the depth of phosphorus penetration in sandy soils would be about 50 cm (20 in) per year while in finer textured soils it may be as low as 10 cm (4 in) per year (Sikora and Corey, 1976). Therefore, groundwater contamination with phosphorus where clean sands are found may become a problem, but only after a considerable length of time.

The fate and significance of heavy metals and complex organic compounds in the soil around absorption systems have not been determined. An insufficient data base exists.

Though the soil does not do a perfect job of treating wastewater, proper design and management of soil absorption systems allows the soil to absorb the liquid and remove a very high percentage of the organisms and substances potentially harmful to human health and the environment.

SECTION 7

ON-SITE TREATMENT AND DISPOSAL ALTERNATIVES

A satisfactory wastewater treatment system discharges a water of acceptable quality which prevents the accumulation of harmful pollutants to dangerous levels in the environment. The environment, of course, may be part of the system, providing the final treatment necessary before the water is of sufficient quality for reuse. If the pollutant load received by the environment is too great, the pollutants will not be broken down and recycled rapidly enough, allowing the pollutants to accumulate. This leads to failure of the system.

The maximum permissible limits of pollutants that can be discharged vary with the type of pollutant and the receiving environment. Therefore, to properly design a wastewater treatment system, it is necessary to evaluate the physical characteristics of the site where the partially treated wastewater is to be discharged. Each site has its own characteristics that limit its potential as a treatment medium. These characteristics will dictate the type and degree of pretreatment that is required.

Proper evaluation of the receiving environment becomes particularly critical where on-site wastewater treatment systems are necessary. On-site systems lack the advantage of central sewerage which collect and convey wastes to a treatment plant located at a site which is selected for its suitability to receive the pretreated wastes. Instead, they must be located near the point of waste generation where local environmental conditions are often less than ideal.

Traditionally, the septic tank-soil absorption system has been used to provide on-site treatment and disposal of liquid systems. Soils are very effective biological and physical filters which break down organic and other chemical substances as well as remove pathogenic organisms and viruses. Where soils are suitable, they should be utilized in the treatment system. However, not all soils and the site characteristics with which they are associated, are equally effective in providing absorption and purification over a reasonable lifetime. If the soil and site characteristics preclude soil absorption for on-site disposal, other alternatives must be sought (see Figure 17).

An on-site wastewater treatment and disposal system must be designed to produce an effluent of sufficient quality to be compatable with the method of final disposal used. One or more unit processes would be placed in series to provide the necessary treatment as shown in Figure 17. The system selected may or may not be dependent upon soil for disposal. For a detailed discussion of these systems, see Appendices A and B.

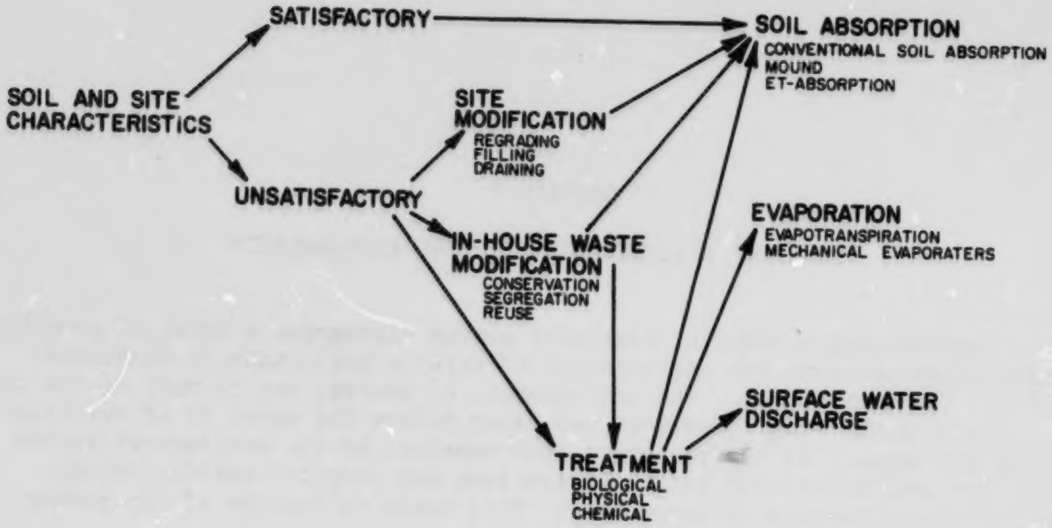

SATISFACTORY
SOIL ABSORPTION
CONVENTIONAL SOIL ABSORPTION
MOUND
ET-ABSORPTION
SOIL AND SITE CHARACTERISTICS
SITE MODIFICATION
REGRADING
FILLING
DRAINING
UNSATISFACTORY
EVAPORATION
EVAPOTRANSPIRATION
MECHANICAL EVAPORATERS
IN-HOUSE WASTE MODIFICATION
CONSERVATION
SEGREGATION
REUSE
SURFACE WATER DISCHARGE
TREATMENT
BIOLOGICAL
PHYSICAL
CHEMICAL

Figure 17. Alternative strategies for on-site wastewater treatment and disposal.

SYSTEMS DEPENDENT ON SOIL

The Conventional Septic Tank-Soil Absorption System

The most common method of on-site liquid waste disposal is the septic tank-soil absorption system. The conventional septic tank system has two components: a septic tank, used to provide partial treatment of the raw waste, and the soil absorption field or pit where final treatment and disposal of the liquid discharged from the septic tank takes place. Both are installed below the ground surface.

The septic tank system has a bad reputation because failures are common. Failure usually manifests itself by seepage of septic tank effluent to the ground surface or by sewage back-ups in the house plumbing due to a clogged soil absorption field. Since the system is near the home or establishment, the seepage is readily accessible to humans and pets. The seepage may also enter surface waterways, increasing the risk of exposure to potential health hazards. A more serious type of failure, however, occurs when there is insufficient or unsuitable soil below the absorption field to properly purify the septic tank effluent before it reaches groundwater. Contamination of nearby wells by bacteria, viruses and chemical pollutants can result. This type of failure often goes unnoticed until illinesses or epidemics occur.

Failure is not due to inherent shortcomings of the septic tank-soil absorption system itself, but rather to misapplication and misuse. Where soils are suitable, the septic tank-soil absorption field is an excellent method of on-site disposal of wastewater. Therefore, site evaluation is a critical step in its design.

Site Evaluation--

Site factors that influence the operation of on-site absorption systems include the soil's hydraulic conductivity, the depth of soil over zones of saturation or bedrock, the slope and topographic position, and the site's management history. Of these, the soil's hydraulic conductivity and unsaturated depth are most important to insure absorption of all the wastewater generated and adequate treatment of the waste before the liquid reaches the groundwater.

Estimation of the Infiltrative and Percolative Capacity of Soil--Direct, on-site measurement of how the soil will respond to continuous wastewater loading cannot be done practically. Instead, equilibrium flow rates through clogged soils usually must be estimated from a short term empirical soil test.

The percolation test--In 1926, Henry Ryon developed a test to determine the relationship between soil type and hydraulic loading of seepage systems (Federick, 1948). He studied both properly functioning and failing systems. He dug a hole one foot square to the depth of the systems, soaked the hole overnight to allow the soil to swell, refilled the hole the following day, and recorded the time required for the water level to drop one inch ("percolation rate"). To calibrate the test, Ryon inspected several failing or near-failing systems and noted the loading of the system, the soil characteristics and the percolation rate measured in nearby soil. Ryon plotted curves relating loading rates versus the percolation rate from these data. It was later proposed that these curves could be used to size new soil absorption systems. Adoption of the procedure by the New York State Health Department led to its wide acceptance, though slight changes have been made over the years. Today it is used by most states to size on-site systems.

The use of the percolation test data for soil absorption system design is based on the assumption that the ability of a soil to absorb sewage effluents over a prolonged period of time may be predicted from the soil's initial ability to absorb clear water (McGauhey and Winneberger, 1963). From Ryon's data comparing absorption rates of existing septic tank systems to the percolation test, it is necessary to reduce the measured rate by an empirical factor ranging from 20 to 2500 in order to size the absorption area (Bouma, et al. 1972). However, tests run in the same soil can vary by several orders of magnitude (Bouma, 1971; Healy and Laak, 1973; Winneberger, 1974). Thus, the procedure is unreliable, and a more accurate test is desirable, or at least, less reliance should be put on its results in system design.

The "crust test"--The soil below properly designed and operating absorption systems is unsaturated because of the clogging mat which develops at the infiltrative surface. To properly size an absorption system, therefore, the unsaturated hydraulic conductivity characteristics of the soil must be known since the unsaturated conductivity is significantly less than the saturated conductivity. Since the standard percolation test does not provide conductivity data of this type, the "crust test" was developed (Bouma, et al 1971, 1972; Bouma and Denning, 1972).

The crust test is performed in situ to avoid disturbing natural pores and to maintain continuity with the underlying soil. A soil column is carved from the soil horizon of interest and fitted with a ring infiltrometer (an impermeable collar with a tight fitting lid) to control water addition to the column. A tensiometer is installed in the column just below the infiltrative surface to determine the degree of saturation in the soil by measuring the soil moisture tension as water is applied (Figure 18). To create unsaturated conditions in the soil column, a "crust" made of gypsum or gypsum and sand is placed over the soil surface. When water is introduced through the infiltrometer, flow into the soil is restricted by the crust. This establishes a constant steady-state flow which induces a nearly uniform moisture tension in the soil beneath the crust. The measured soil moisture tension and the equilibrium flow rate for a given crust determines one point on the hydraulic conductivity curve. Additional tests run on the same column with crust of different hydraulic resistances establish points that define a K-curve as shown in Figure 9. This curve can be used for design if the range in soil moisture tensions under the clogged zones of mature absorption systems in similar soils is known.

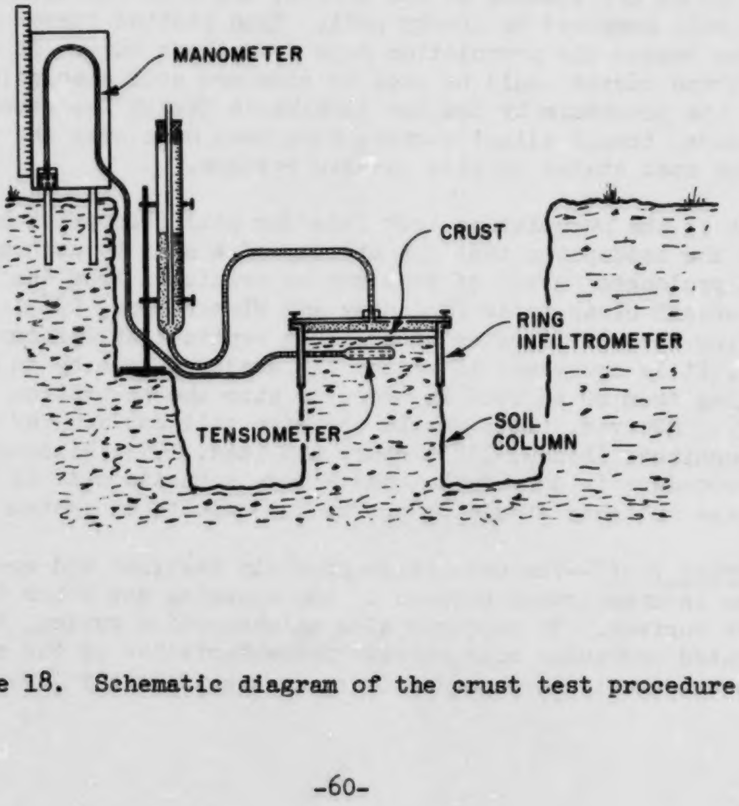


Figure 18. Schematic diagram of the crust test procedure.

Though this procedure offers a direct measurement of K, it is time consuming and requires a skilled operator. It is not a test which can be run economically at each site. However, since the hydraulic conductivity of a soil is dependent upon the pores in the system, the conductivity of a soil at various sites in the soil map unit can be defined within statistical limits (Figure 19). Also, it has been found that curves of different soils in the same textural groups are similar at moisture tensions greater than 10 mbar (Figure 20). Therefore, by defining families of K-curves for groups of soils, the hydraulic conductivity characteristics of a particular soil or site can often be predicted without the need and expense of on-site testing.

In Wisconsin, four major hydraulic conductivity types have been suggested based on the texture of the soil materials (Bouma, 1975). These textural groupings include the sands; sandy loams and loams; silt loams and some silty clay loams; and the clays and some silty clay loams. In other regions similar groupings might be made, but they must be based on field data since differences in soil mineralogy may affect these groupings. Typical hydraulic conductivity curves were developed from field measurements for each of these conductivity types (Figure 9).

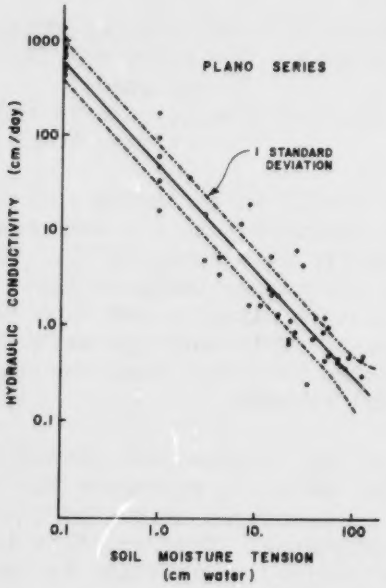


Figure 19. Hydraulic conductivity data for Plano series Regression line is solid line, and dashed lines indicate one standard deviation about regression line.

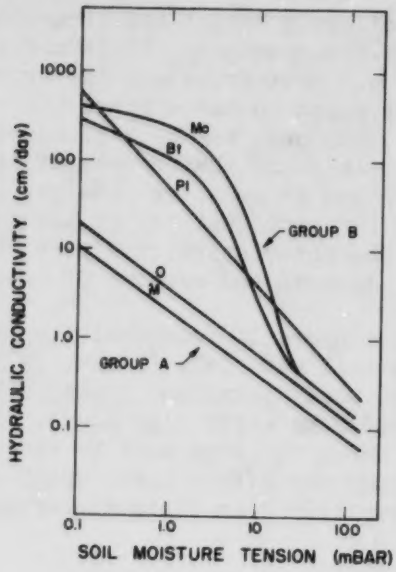


Figure 20. Hydraulic conductivity groups:
Group A: Ontonogon and Magnor series
(heavy loams)
Group B: Plano, Batavia and Morely series
(silty clay loams)

To make these curves useful in designing soil absorption fields for septic tank systems, soil moisture tensions were measured under the clogging zones of several operating fields (Bouma, 1975). This information provided a design point on the curve for proper field sizing. For example, the soil moisture tension below a mature clogging mat in a silt loam soil is expected to be between 20 and 34 mbar. This corresponds to a K of from .67 to .92 cm/day (Bouma, 1975). This same procedure could be used to select design points for the other types of soil systems.

The application rates for various soil conductivity types defined in Wisconsin and presented in Table 23, represent the best estimates available to date. Because of the unstructured nature of the sands and sandy loams, the rates are reasonably accurate. However, flow through finer textured soils is more complex and there is more variability in the tensions measured under operating fields (Bouma, 1975). In these soils the design rates must be used with care particularly if expandable clays are present.

With no reasonably simple alternative to determine the equilibrium infiltration rate of soils under wastewater application, the percolation tests continues to be favored. However, other information such as soil texture and structure should be used to supplement and confirm the test.

Estimation of the unsaturated depth of soil--To insure adequate purification of the wastewater before it reaches groundwater, three feet of unsaturated soil is necessary below the infiltrative surface. If saturated soils ever occur within the three feet minimum, transmission of harmful pollutants to the groundwater may result (Green and Cliver, 1974; McCoy and Ziebell, 1975; Ziebell, 1975). To determine if saturated conditions do occur within the minimum is often difficult, however, because water table levels fluctuate in response to changing weather conditions. Typically, the water table is low during the summer, while in the spring and fall, it rises. Ideally, the highest groundwater level should be observed when it occurs, but this is not always practical. Moreover, observations made in relatively dry years do not represent those that occur in normal years. Thus, other methods must be used to determine the high water elevation.

Soil mottling is sometimes an indicator of the presence of seasonally high water levels. Mottles are spots of contrasting colors found in soils subject to periodic saturation. The spots are usually bright yellow-orange-red surrounded by a gray-brown matrix and described according to their color, frequency, size and prominence (Soil Survey Staff, 1951). Well-drained soils are usually brown in color due to the presence of finely divided insoluble iron and manganese oxide particles distributed throughout the horizon. However, under reducing conditions often produced by saturation over prolonged periods, the iron and manganese is mobilized until reoxidized when the soil drains. Repetitive wetting and drying cycles quickly produce local concentrations of these oxides on pore surfaces forming red mottles (Vepraskas and Bouma, 1976). Soil from which much of the iron and manganese has been completely reduced, loses its brown color, and becomes grey by a process referred to as gleying. Therefore, the upper limit of the mottled soil is often a good estimate of the high groundwater level.

It is possible for soils to saturate and not develop mottles. In regions where the water remains cold or bacterial activity is limited, mottle formation is hindered. Also, soils with high pH may not develop pronounced mottling. This is true of the red clay soils of Northern Wisconsin which saturate but do not exhibit mottling.

Sizing of infiltrative surface--Proper design of the soil absorption field requires that the rate of wastewater application to the infiltrative surface not exceed the soil's equilibrium infiltration rate. The equilibrium infiltration rates have been determined for the various hydraulic conductivity soil types in Wisconsin (Table 23), so it remains to estimate the total daily volume of wastewater to be discharged to the field.

Waste flows from single homes, restaurants, motels, etc., are intermittent and subject to wide fluctuations. Variation in the number of persons contributing to the flow and their activitieis have profound effects on the daily volume of waste generated. Therefore, accurate estimates of waste flow volumes are difficult. Estimates of per capita contributions from single homes are presented in Table 4. More detailed estimates of household and commercial wastewater flows are discussed in Appendix A.

Because the flow must be contained in a limited area, soil absorption fields must be designed for maximum rather than average daily flow unless provisions are made for flow equalization. It is common practice to size the infiltrative surface based on the maximum potential use of the building to be served rather than its initially intended use. For example, to size a system for a household in a soil with a percolation rate of 12 min/cm (30 min/in), the Manual of Septic Tank Practice (USPHS 1967) recommends an absorption area of 22.8 m^2 (250 ft^2) per bedroom. This is based on a waste flow of 568 L/d (150 gpd) per bedroom assuming two occupants per bedroom each generating 284 L/d (75 gpd). Thus, for a three bedroom home, an area of 68.4 m^2 (750 ft^2) is required regardless of the number of people who reside there.

A better method of sizing would be to use the soil's equilibrium infiltration rate for septic tank effluent (Table 23) and an accurate estimate of the maximum daily wastewater volume that can be generated by the fixtures installed in the building. If designed in this manner, allowances for differences in plumbing fixtures used, the potential use of the building, etc., could easily be made.

Geometry of the Infiltrative Surface--

Bottom versus sidewall area--Both the horizontal bottom area and vertical sidewalls of a subsurface soil absorption system can act as infiltrative surfaces for wastewater absorption. When a system is first put into operation, the bottom area is the only infiltrative surface. However, after a period of wastewater application, this surface can become clogged sufficiently to pond liquid above it, at which time the sidewalls become infiltrative surfaces. Because the gradients and resistances of the clogging mats at the two surfaces are rarely the same, the infiltration rates will be different. Which surface would have the greatest infiltration rate will depend on a number of factors. Vertical and horizontal hydraulic conductivities and gradients in the soil, clogging mat resistances, and soil moisture contents of the surrounding soil are factors that will effect the direction and rate of liquid movement through the soil. Thus, the more significant infiltrative surface may vary with time and between sites. The objective in design, therefore, is to maximize the area of the surface expected to have the highest flow rate. Based on investigations done at the University of California in Berkeley, McGauhey and Winneberger (1965) concluded that the sidewall is "... by far the most effective infiltrative surface." They reasoned that 1) suspended solids in the effluent do not contribute to sidewall clogging, 2) rising and falling liquid levels within the system allow alternate loading and resting of the surface while the bottom is continuously inundated, and 3) sloughing of the clogging mat can occur during resting periods. Therefore, they recommend that subsurface soil absorption systems should provide a maximum of sidewall surface per unit volume of effluent and a minimum of bottom surface.

The extent to which the sidewall becomes an infiltrative surface would depend upon the prevailing hydraulic gradients which is largely determined by the soil type and soil wetness surrounding the system. At the bottom surface, gravity, the pressure of the ponded water above, and the matric potential of the soil below, all contribute to the total hydraulic potential of the liquid while at the sidewall, gravity does not contribute to the potential gradient since it operates vertically and the pressure potential diminishes to zero at the liquid surface. In temperate climates, frequent rainfall, particularly in the spring and fall, may reduce the matric potential at the sidewall to low levels due to percolating precipitation. During such times, the horizontal gradient could be significantly less than the vertical gradient with the effect that the bottom surface would become the dominant infiltrative surface. For this reason, Bouma (1975) recommends that in temperate climates, systems should be sized on bottom area only. Healy and Laak (1974) do not suggest that a system be designed on bottom area only, but they do recommend that in temperate zones, systems be designed to function under gravity potential only because of the problem during wet portions of the year. They also state that evapotranspiration during such times is too low to remove significant volumes of wastewater because of the wet soil. The ability of the soil to transport the liquid to the surface for evapotranspiration, of course, is directly related to the matric potential or "wicking" action of the soil.

If the trenches were to remain ponded, deep narrow trenches could be constructed to increase the hydraulic gradient across the sidewall, as recommended by McGauhey and Winneberger (1965). However, this would diminish the advantages of shallow trenches which enhance the potential for evapotranspiration and avoid construction in the deeper soil horizons where puddling and compaction are more likely due to wet finer textured soil. It might be concluded that in humid regions, systems should be designed on bottom area while maximizing the sidewall by utilizing shallow trenches rather than beds. In more dry regions, with rather permeable soils, the sidewall area could be maximized at the expense of the bottom area.

Trench versus bed design--Though beds often are more attractive than trenches because total land requirements, cost, and time of construction are less, trenches are more desirable in terms of maintaining the infiltrative and percolative capacity of the soil. This is particularly true in soils with significant clay contents (>25 percent by weight). The principal advantages of trenches over beds are that: 1) more infiltrative surface is provided for the same bottom area, and 2) less damage is likely to occur to bottom infiltrative surface due to compaction, puddling and smearing during construction.

For identical bottom areas, trench designs of absorption fields can provide more than 8 times the sidewall area. This can be of benefit in preventing failure through clogging. In humid climates, there may be portions of the year that the sidewall loses much of its effectiveness for absorption which necessitates designing the system to function on bottom area only. However, it is recognized that the sidewall is beneficial and it is certainly recommended to maximize it in any system (Bouma, 1975; McGauhey and Winneberger, 1975).

In addition, the seepage bed design can cause severe damage to the natural soil structure during installation. This is a particular concern in clayey soils. Rapid absorption of liquid by the soil depends on a suitable soil structure being maintained (Bouma, 1975; Bouma,et al 1975a). When mechanical forces are applied to moist or wet soil, the structure is partially or completely destroyed because clay particles in the soil are able to slip relative to one another. This movement, which results in compaction, puddling or smearing, closes the larger pores between soil aggregates and those made by roots, or burrowing soil fauna.

To construct a seepage bed, it is common practice to first scrape off the topsoil using a front end loader and then return with a backhoe for digging to final grade in an attempt to leave a fresh soil surface. However, these two operations may require several passes over the bed area by the construction machinery often with heavy loads. When digging is complete, trucks may be backed into the bed to unload aggregate which is spread over the bottom of the bed with machinery. After the distribution piping is laid, additional gravel is placed over the pipe and covered with soil. By the time the bed is completed, the soil structure may be destroyed.

This problem is further compounded when soil conditions are wet. A busy contractor is unable to always schedule his work when the soil is dry, so construction often proceeds when conditions are marginal at best. The trench design reduces the severity of these problems because the construction machinery is able to straddle the trench so that the future infiltrative surface is never driven upon.

To make trench systems more favorable, design codes should encourage the use of trenches. A reasonable approach would be to require more bottom area for beds than trenches for the same size household. Two methods might be used: 1) give credit for sidewall area thereby reducing the bottom area required for trenches, or 2) increase the bottom area now required for beds in proportion to the amount of sidewall area lost by not using the trench design. Before the former approach is recommended, however, more needs to be learned about the relative contributions of the sidewall and bottom areas as infiltrative surfaces.

Shallow versus deep absorption systems--Shallow soil absorption systems offer several advantages over deep systems: 1) the upper soil horizons are usually more permeable than the deeper subsoil because of greater plant and soil fauna activity and eluviated clay, 2) evapotranspiration is greater, 3) the upper soil dries quicker than the subsoil so construction can proceed over longer periods of the year with less smearing, puddling and compaction, and 4) less excavation is necessary, reducing the cost. Some state codes prohibit the construction of absorption systems deeper than 90 cm (36 in). This restriction seems reasonable but only if more permeable soil horizons do not exist at greater depths. In such instances, deep systems may be practical where the groundwater tables does not preclude their use.

Freezing of shallow absorption systems is not a problem if kept in continuous operation even when frost penetration is quite deep. Weibel, et al. (1949) reviewed the literature and made contacts with health authorities and plumbers in the northern states to determine if failures of shallow systems were frequent due to freezing. They concluded that carefully constructed shallow systems 45 cm to 60 cm (18 in to 24 in) in depth would not freeze even in areas where frost penetration reaches 1.5 m (5 ft), if the tile lines were gravel packed and header pipes insulated where it is necessary for them to pass under driveways or other areas usually cleared of snow.

Distribution of Liquid Over the Infiltration Surface--

To insure that the objectives of absorption and treatment are met over a long system lifetime, the method of wastewater application to the infiltrative surface must be compatible with the local soil and site characteristics. That is, suitable unsaturated conditions must exist for at least 90 cm (3 ft) below the infiltrative surface at all times without excessive clogging occurring. There are three basic methods of wastewater application for which a distribution network can be designated: 1) continuous ponding, 2) dosing and resting, and 3) uniform application without ponding.

Uniform application without ponding would seem to be the best loading method for most soil and site conditions. In rapidly permeable soils, this method is essential to insure adequate treatment during initial operation when no clogging mat is present to prevent short-circuiting (Bouma, 1975). In fine textured soils, however, it may not be possible to maintain a loading regime or uniform application without ponding. A system designed to utilize this method of application may revert to a continuously ponded regime due to excessive clogging. Continuous ponding may be necessary in fine textured soils to provide the necessary gradient across the clogging mat to absorb all the wastewater. More research is needed to make this determination.

Dosing and resting loading regimes may be appropriate where absorption is the principal concern. This is true only if dosing and resting, either on a daily schedule or on a monthly or yearly schedule using alternating absorption systems, actually retard clogging. Again, more research is needed. Limited laboratory and field data seem to be contradictory on this point. Dosing and resting systems probably should not be used in highly permeable soils with a high water table unless small, frequent doses are applied each day (Bouma, 1975). Long periods of aeration would permit the clogging mat to degrade, thus allowing pollutants to penetrate to the ground water, as is the case during initial operation of a system where no clogging mat is present.

Site limitations may be present which require loading methods that spread the wastewater over a large area. Examples of these site limitations are high ground water or shallow, impermeable bedrock or cemented pan where groundwater mounding may occur to reduce the unsaturated depth of soil. Uniform application without ponding would be best, although dosing and resting may be suitable if the dosing volume necessary to pond the whole infiltrative surface is not excessive. On steeply sloping sites, uniform application without ponding would be the most appropriate to prevent seepage downslope. Very stony or gravelly sites should be avoided unless suitable filtering material is brought in.

Distribution network designs--Many different network designs have been used in soil absorption systems all with the intent of uniformly applying liquid over the entire infiltrative surface. This rarely is achieved, but it may not always be necessary. The designs include: large diameter perforated pipe networks and pressure distribution networks. The choice of one over the other depends upon the loading regime desired.

Large diameter perforated pipe networks--The conventional distribution network for soil absorption fields consists of perforated 10-cm (4-in) diameter pipe through which the liquid flows by gravity. The pipe is laid level or on an 0.17 to 0.33 percent slope. In multi-trench or bed systems, the pipes are interconnected by a common solid wall header pipe, drop box or distribution box.

The purpose of laying the pipe on a true, prescribed slope is to get uniform distribution as the effluent trickles or flows in by gravity. However, this is not the case, McGauhey and Winneberg (1964) and Bouma, et al. (1972) observed nonuniform distribution. As it flows into the pipe, effluent seems to exit out of a few holes either at the inlet area, middle or far end of the trench. This causes localized overloading where small areas receive a more or less continuous trickle of effluent. At first, adequate treatment by the soil is not achieved because saturated flow conditions are created. Soon, biological clogging occurs and reduces the infiltration rate below the rate at which effluent is discharged. The effluent is forced to flow along the bottom of the trench or bed until it reaches an unclogged area. This phenomenon, known as "creeping failure," continues until the total bottom area of the system is clogged (Figure 21). Altering the orientation of the holes or changing the slope of the pipe does not improve distribution significantly (Converse, 1974).

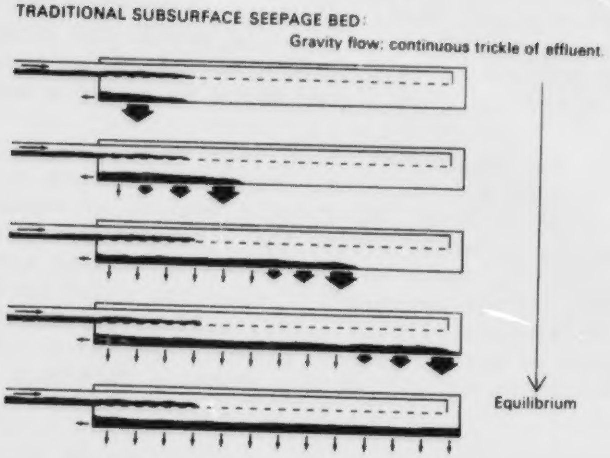

TRADITIONAL SUBSURFACE SEEPAGE BED:
Gravity flow; continuous trickle of effluent
Equilibrium

Figure 21. Progressive clogging of the infiltrative surfaces of subsurface absorption systems (Bouma, et al. 1972).

Periodic pumping of large volumes of effluent onto the field improves distribution and provides an opportunity for the soil to drain between applications. Drainage exposes the infiltrative surface to air, reducing clogging (Bouma, et al. 1975a; McGauhey and Winneberger, 1963, 1964). However, even with dosing, the effluent is not distributed over the entire infiltrative surface if the 10-cm (4-in) pipe is used (Converse, 1974).

Large diameter perforated pipe networks are best suited for continuously ponded or dosing and resting loading regimes. For dosing applications, the critical factor is to discharge by pump or siphon a sufficiently large volume of liquid with each dose to submerge the entire infiltrative surface.

Pressure distribution networks--For uniform application of the wastewater over the infiltrative surface, the distribution network must be designed such that the volume of water passing out every hole within the network is identical. This design permits much better control of application rates and prevents local saturated conditions.

This is most easily done by putting the network under pressure and sizing the pipe and hole diameters to balance the headlosses to each hole. Rules of thumb used are: 1) to assume at least 60 to 90 cm (2 to 3 ft) of head at the terminal end of each lateral, 2) to assume that 65 to 85 percent of the total headloss in the network occurs crossing the orifice, and 3) to assume that 10 to 15 percent of the total headloss occurs in delivering the liquid to each hole. The remaining headlosses would occur through fittings (Otis, et al. 1977). These systems combine uniform distribution with dosing, which enhance purification by promoting unsaturated flow and may reduce clogging.

Proper loading of permeable soils to prevent saturated flow is vital to insure purification of the waste effluent. Pressure distribution systems provide this loading control. Conventional gravity distribution is ineffective (Converse, et al. 1974). Pressure distribution systems also retard clogging. Since the network is designed to apply no more liquid than an area of the absorption bed can absorb each day, the soil remains well aerated. Absorption fields in sand with pressure distribution have shown no evidence of clogging after four years of operation (Converse, et al. 1974), while fields in sand with conventional distribution begin to clog after six months (Bouma, et al. 1972). The aerobic environment maintained by pressure systems promotes the growth of microorganisms which destory clogging materials and appears to attract larger fauna, such as worms, to consume nutrients accumulating at the infiltrative surface. The worm's burrow help break up the clogging zone. Worm activity explains why an absorption field in a silt loam underlain with glacial till and dosed through a pressure distribution network at three time the USPHS (1967) recommended rate has not clogged after three years of operation (Bouma, et al. 1975a).

Pressure distribution networks may be the only alternative where rapidly permeable soils are used for absorption in areas where groundwater contamination is possible. Further field demonstrations are necessary to determine their value in other settings.

Restoring the Infiltrative Capacity of a Clogged Absorption Field--
Soil absorption systems often fail after several years of satisfactory service because the clogging zone eventually develops to a point where insufficient amounts of effluent pass through it. Methods are being sought to rejuvenate old fields so that failed systems need not be replaced.

Resting--One effective method of restoration is resting the system (Bendixen, et al. 1950; Bouma, et al. 1972; McGauhey and Winneberger, 1963; Weibel, et al 1954). Resting allows the absorption field to gradually drain, exposing the clogged infiltrative surface to air. After several months, the clogging materials are broken up through physical and biochemical processes, restoring the infiltrative capacity of the system. This requires a second bed be available to allow continued use of the disposal system while the clogged bed is resting. Two beds can be constructed when the disposal system is first installed at the outset, with an alternating valve located after the septic tank as shown in Figure 22. The two beds can then be use alternately by diverting the wastewater from one to another at an appropriate time interval. If a system with only one bed has failed and a new one is constructed, provisions should be made such that the old one is not abandoned, but can easily be alternated with the new bed by use of an alternating valve arrangement.

Oxidizing agents--The infiltrative surface also can be rejuvenated by the addition of oxidizing agents to the absorption field. The oxidizing agents perform the same function as resting but the clogging zone is destroyed within a day or two rather than several months. Such a method does not necessitate taking the clogged bed out of service which eliminates the need for a second bed.

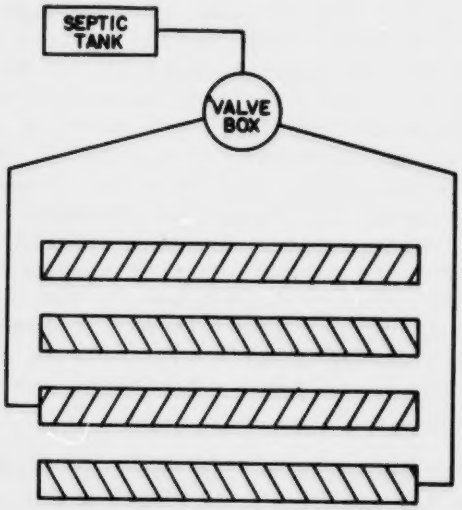


Figure 22. An alternating soil absorption field design.

Laboratory and field tests indicate that chemical oxidation can restore the infiltrative surface to near its original permeability (Harkin, 1975). The oxidant preferred is hydrogen peroxide (H_2O_2) because it is effective at the natural pH of absorption fields, produces no noxious byproducts and is inexpensive. Eight to 160 L (20-40 gal) of a 50 percent H_2O_2 solution has been found to be adequate for rejuvenating conventional soil absorption systems in sandy soils serving single households. The use of this treatment needs further demonstration, particularly on systems in finer textured soils. Its usefulness is limited, however, since it treats only the symptoms of failure and not the causes. It may serve best as a preventive maintenance measure.

The Mound System

There are many areas where the conventional septic tank-soil absorption field is not a suitable system of wastewater disposal. For example, sites with slowly permeable soils, excessively permeable soils, or soils over shallow bedrock or high groundwater do not provide the necessary absorption or purification of the septic tank effluent. However, these limitations often can be overcome by constructing the soil absorption field above the natural soil in a mound or medium sand fill (Figure 23).

There are several advantages to raising the soil absorption field. The fill below the absorption trenches within the mound provides additional soil material necessary to purify the wastewater before it reaches the groundwater at sites with shallow or excessively permeable soils. At sites with slowly permeable soils, the purified liquid is able to infiltrate the more permeable natural topsoil over a large area and safely move away laterally until absorbed by the less permeable subsoil. Also, the clogging mat that eventually develops at the bottom of the gravel trench within the mound will not clog the sandy fill to the degree it would in the natural soil. Finally, smearing and compaction of the wet subsoil is avoided since excavation in the natural soil is not necessary.

The design of the mound is based upon the expected daily wastewater volume it will receive and the natural soil characteristics (Converse, et al. 1975a, 1975b, 1975c). It must be sized such that it can accept the daily wastewater flow without surface seepage when perched water exists in the natural soil in the spring and fall, as well as when the water table is lower during the summer and winter. Size of the seepage trenches or bed and spacing of the seepage trenches is important to avoid liquid rising into the fill below the seepage area when the water table is high. In addition, the total effective basal area of the mound must be sufficiently large to conduct the effluent into the underlying soil.

A clean, medium sand is used as the fill material in construction of the mound and gravel is used in the trenches. As in any seepage trench, a clogging mat will develop at its interface. The ultimate infiltration rate through this zone has been shown to be 5 cm/day (Bouma, 1975). Therefore, one consideration must be to insure that sufficient trench area is available for the design flow. As in conventional system sizing, bottom area is used for design, and sidewalls constitute a safety factor.

If more than one trench is constructed within the mound, another consideration is the spacing between trenches. The area between trenches must be sufficient for the underlying natural soil to absorb all the liquid contributed by the upslope trench. Infiltration rates into the natural soil are based on the hydraulic conductivity characteristics of the least permeable soil horizon 90 cm (3 ft) below the proposed site. The basal area required for the mound is based on this as well.

To distribute the wastewater to each of the trenches, a pressure distribution network is used. This provides uniform application which is necessary to prevent local overloading and eventual surface seepage due to short circuiting through the mound fill.

Mound systems have been installed and monitored since 1972 and are performing satisfactorily (Bouma, et al. 1973, 1975b). However, application of proper siting, design, and construction techniques, described in detail by Converse, et al. 1975a, 1975b, 1975c, are critical for satisfactory performance.

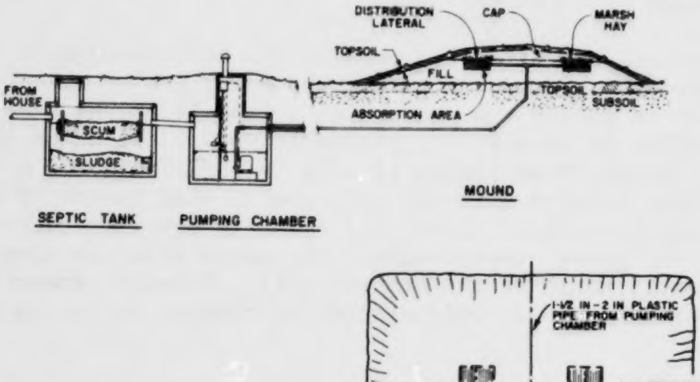

DISTRIBUTION LATERAL
CAP
MARSH HAY
TOPSOIL
FILL
TOPSOIL
SUBSOIL
ABSORPTION AREA
FROM HOUSE
SCUM
SLUDGE
MOUND
SEPTIC TANK
PUMPING CHAMBER
1-1/2 IN - 2 IN PLASTIC PIPE FROM PUMPING CHAMBER
1 IN PERFORATED PLASTIC PIPE
SEEPAGE TRENCH 3/8 IN - 1 IN STONE
PLAN VIEW

Figure 23. A plan view and cross-section of a mound system for problem soils.

Based on the success of the experimental mounds, their use for on-site disposal are recommended for sites with: 1) slowly permeable soils with percolation rates at 60 cm (24 in) of 0.02 cm/min (120 min/in) or faster, 2) permeable soils with percolation rates at 30 cm (1 ft) faster than 0.04 cm/min (60 min/in) over creviced or porous bedrock of 60 cm (2 ft), and 3) permeable soils with percolation rates of 60 cm (2 ft) faster than 0.04 cm/min (60 min/in) with water tables of 60 cm (2 ft). The three mound designs have been adopted in several states.

SYSTEMS NOT DEPENDENT ON SOIL

At some sites, the soils may be totally inadequate as a treatment and disposal medium. In such instances, on-site wastewater treatment systems not dependent upon soil disposal, but which discharge the treated wastewater to surface waters, to shallow soils overlying creviced bedrock or high water tables, or to the atmosphere, are necessary.

Systems which must be designed to meet a certain water quality objective may incorporate a variety of treatment processes discussed earlier, yet only a select few will prove to be both economically and environmentally acceptable. The selection process involves the evaluation of technical feasibility, cost effectiveness and administrative feasibility. A systematic procedure is required to evaluate all physical constraints which may influence the selection of treatment or disposal options. In all cases, an appropriate institutional framework must be developed to insure appropriate construction, operation and maintenance of the system.

Three experimental field systems which employed a selection of sequences of processes were evaluated in this study over a two year period. The details of the studies at field sites E, H and J appear in Appendix A. Figure 24 depicts the flowsheets employed at these three field sites. A summary of the performance of these three sites is presented in Table 24. Data in Table 24 has been extracted from Tables A-120, A-121, A-131, A-136 and A-139. It is apparent from studying this summary tabulation that effluent qualities from these three systems exceed current environmental quality standards for the measured pollutants except for the nutrients, nitrogen and phosphorus, where they are limited. Operation and maintenance characteristics of the systems studied are delineated in Appendix A. Costs may be synthesized from data presented in Section 9.

Site E

Septic Tank Tank → Sand Filter → Chlorination →

Site H

Septic Tank → Aerobic Unit → Sand Filter → Chlorination →

Site J

↗ Sand Filter → UV Disinfection → Denitrification →
Septic Tank
↘ Aerobic Unit → Sand Filter → UV Disinfection → Denitrification →

Figure 24. Flowsheets for three on-site field systems

TABLE 24. EFFLUENT QUALITY FOR THREE FIELD SYSTEMS

Parameter	SITE F					SITE H					SITE J						
	Septic Tank	Sand Filter	Chlorination	Septic Tank	Sand Filter	Aerobic Unit	Sand Filter	Chlorination	Aerobic Unit	Sand Filter	Septic Tank	Sand Filter	UV Disinf.	Denitrification [2]	Aerobic Unit	Sand Filter	UV Disinf.
	Aug 1974-March 1975			March 1975-Dec 1976		Aug 1973-Feb 1975			Mar 1975-May 1976		June 1976-Dec 1976				June 1976-Dec 1976		
Filter Loading Rates m/d (gal/ft^2/d)	.19-.21		(4.8-5.3)	.09-.34	(2.2-8.4)	0.11-0.16		(2.7-4.0)	.04-.29	(1-7.3)		0.11-0.16	(2.8-3.0)		0.11-0.16		(2.8-3.3)
BOD$_5$ (Unfilt.), mg/L																	
Mean	123	9[1]		109	7.1[1]	26	3.5[1]		43	3.3[1]	57	1.6[1]		-	6	1.2	
95% Conf. Int.	81-165	6-14		73-145	4.8-10	21-31	2.7-4.5		30-65	2.2-5.7	47-66	1.3-1.9			2-10	1.0-1.4	
Suspended Solids, mg/L																	
Mean	48[1]	7[1]		39[1]	2.9[1]	48[1]	11[1]		23[1]	6.4[1]	34[1]	14[1]		-	12	3.9	
95% Conf. Int.	26-89	3.9-12		26-57	1.3-6.3	39-58	8.5-14		16-32	3.2-13	29-40	11-18			6-17	2.8-5.4	
Volatile Suspended, mg/L																	
Mean	37	5		28[1]	1.4[1]	33	5.3[1]		15[1]	2.8	28[1]	5[1]		-	5	2.0[1]	
95% Conf. Int.	21-63	3-8.2		17-45	0.7-2.8	27-41	4.3-6.5		9-24	1.6-4.9	23-33	4-6.2			3-7	1.3-3.0	
Total Nitrogen, mg/L																	
Mean	26	24		27	33	39	36		31	29	36	-			31	-	
95% Conf. Int.	17-34	17-33		10-45	20-46	35-43	32-40		24-39	21-37	34-37	-			30-32	-	
Ammonia-N, mg/L																	
Mean	21	0.9[1]		19	5.5	0.4	0.3		7	0.5	26	<0.1		<0.1	<0.1	<0.1	
95% Conf. Int.	15-26	-		7-30	2.4-8.6	<0.1-1.6	-		0.2-14	<0.1-1.1	22-29	-			-	-	
Nitrate-N, mg/L																	
Mean	0.3	20.9[1]		0.1	32	34	35		22	28	0.2	34		3.7	30	30	
95% Conf. Int.	-	10-40		-	14-50	30-38	31-39		9-34	19-37	-	31-37		-	30-31	29-32	
Total Phosphorus, mg/L																	
Mean	11	9[1]		12	16.5	32	21		17	17	16	15		14	9	8	
95% Conf. Int.	8-14	7-11		8-16	12-21	24-39	17-26		14-20	11-24	15-17	14-16			8-10	6.4-8.6	
Orthophosphrous, mg/L																	
Mean	10	7[1]		8	10	28	21		10	9	15	13		12	8	7	
95% Conf. Int.	7-13	5.4-9.0		5-12	7.8-12	23-33	17-25		8-13	5-13	13-16	12-14			7-9	6.2-7.0	
Total Coliforms, log #/L																	
Mean	7.0	3.9	1.5	6.4	4.0	-	5.2	2.5	6.2	4.4	7.0	4.4	1.0		4.4	2.4	<1.0
95% Conf. Int.	6.7-7.2	2.2-5.6	0.7-2.2	5.7-7.2	2.7-5.2	-	4.7-5.7	2.0-3.1	5.3-7.1	3.7-5.0	6.7-7.3	4.2-4.6	-		4.2-4.6	2.0-2.6	-
Fecal Coliforms, log #/L																	
Mean	6.8	3.9	1.3	6.3	3.9	5.3	4.0	1.9	5.2	3.9	6.8	4.2	.9		4.1	1.8	<1.0
95% Conf. Int.	6.6-7.0	1.6-6.2	0.7-2.1	5.4-7.2	3.2-4.6	5.1-5.4	3.3-4.6	1.5-2.3	4.0-6.4	2.9-4.8	6.6-7.0	3.9-4.5	-		3.1-4.4	1.0-2.8	-
Fecal Streptococci, log #/L																	
Mean	4.5	2.4	1.3	5.0	2.4	4.4	3.2	1.9	4.4	3.2	5.2	2.9	.8		3.4	2.0	0.8
95% Conf. Int.	4.1-4.8	1.0-5.2	0.9-1.8	4.0-6.0	1.6-3.2	4.2-4.6	-	1.5-2.2	2.5-5.9	1.6-4.7	4.8-5.7	2.4-3.4	-		2.9-3.8	1.2-2.7	-

[1] Log normal distribution.

[2] Field denitrification June-Oct. 1976.

SECTION 8

MANAGEMENT OF ON-SITE WASTEWATER DISPOSAL SYSTEMS

Improved techniques for design and installation of on-site wastewater disposal systems can have a widespread effect on public health, but it will put additional burdens on the regulatory agencies to insure that proper design and construction procedures are used. The lack of effective administration and regulation by government agencies has been responsible for some conventional system failures. The availability of alternate systems will intensify this problem. Despite differences between the conventional and alternative systems, their administration and regulation should be similar. Present regulatory schemes used by states and counties can be adapted to provide better regulation of all types of systems. A more detailed discussion of this topic is presented in Appendix D.

REGULATORY AUTHORITY OPTIONS

Programs for regulating on-site disposal systems varies widely from state to state and among local authorities (usually local units of government or health authorities). The programs used vary from no state or local regulation whatsoever to almost total state regulation of all on-site systems. Some programs share responsibility between the local and state authorities for setting standards, inspection, permit issuance, and enforcement.

The various regulatory prgorams can be categorized into four general types (Stewart, 1974a). First, many states require a state permit for on-site systems and a site inspection by a state agent (Patterson, et al., 1971). This is an effective approach because the pressure often put upon enforcement officials, which can weaken a regulatory program, is usually not as effective at the state level as at the local level.

Second, a few states have no regulation program at either the state or local level (Patterson, et al., 1971). Some of these states supply the public with information on system design, but only for public education purposes. Some of these states will take regulatory action against on-site systems only if a water pollution or health violation is perceived.

Third, some states defer all permit and inspection responsibilities to county or town government or health authorities (Patterson, et al., 1971). In some of these states, a state code of minimum standards and specifications for on-site systems have been adopted requiring local codes and standards to be at least as stringent as the state codes.

Fourth, some states divide responsibilities with the local authorities regulating single family disposal systems and the state regulating all subdivisions and commercial installations. For example, the State of Wisconsin issues construction permits and inspects all systems serving public buildings such as theaters, assembly halls, schools, apartment buildings, hotels, prisons, factories, mobile homes, camps and parks while county authorities regulate all one and two family systems.

Standards and specifications for installation of on-site systems used by the 50 states also vary. Many states base their standards on the U.S. Public Health Service's Manual of Septic Tank Practice (1967). However, other states and localities have developed standards of their own.

REGULATION AND CONTROL

A regulatory program should control the installation, operation and ultimate failure phases of all on-site wastewater disposal systems (Stewart, 1974a, 1974b). Within each phase, one or more problems can arise where regulation can help to prevent public health hazards from occurring.

System Phases Requiring Regulation

Installation of on-site systems includes site selection, design and construction. The type of disposal system installed and design criteria used are determined by the site. The regulatory program must insure that the proper system is chosen and the installation criteria followed.

Operation of the disposal system includes regular and proper maintenance. This must be assured to prevent premature failures. For a conventional septic tank system, this involves inspecting the tank and pumping it when necessary. With alternative systems, operation and maintenance requirements may be more extensive. The regulatory program must insure that appropriate operation and maintenance requirements are met.

Failure of the system may ultimately occur. The regulatory agency should take action to detect failures and to obtain their correction. This involves an assessment of the failure and analysis of alternative corrective measures permitted by the regulations, followed by what can be construed as an installation phase of activity.

Inspections and Permits

Most agencies employ on-site inspections and permit or license issuance to regulate disposal systems. Inspections are generally made during installation. Many regulatory programs only require a "pre-cover up" inspection of the completed system. To be more effective, several inspections should be made including one of the proposed site prior to approving the installation. Like inspections, permits generally are associated only with the installation phase. Good regulatory programs usually require at least two permits for each system. The first typically authorizes construction of the proposed system on a designated site, and is issued after application by the homeowner following inspection of the site or examination of soils and site data. The second permit, referred to as a use/occupancy permit, is typically issued after construction and final inspection of the system is made which authorizes the owner to use the system. Possible uses of inspections, permits and other functions to regulate installation of on-site systems are shown in Table 25.

Inspections and permits also can be used to regulate operation and correct failures. For example, a revocable permit to use the system may be used to insure that septic tanks are pumped. Mandatory regular inspections could be a pre-requisite for the re-issuance of this "operational" (maintenance) permit. Other examples of inspections and permits use are discussed in the following section.

SUGGESTED IMPROVEMENTS FOR REGULATORY PROGRAMS

Suggested improvements for regulation of on-site systems have been made based on a review of regulatory schemes used by many states (Stewart, 1974a, 1974b). These suggestions are not applicable to every state nor are they possible for all states to adopt because of constitutional limitations and requirements. Some of the methods summarized below, however, could be employed by many agencies to improve their regulatory programs.

Installation

1. State permit program. A state agency would receive applications for all on-site systems, make inspections and issue permits for installation and/or use of disposal systems to reduce local political pressure on authorities to approve systems on unsuited sites.

2. State plan review/state standards. Alternatively, a state agency would review all or a representative sample of all on-site permits issued by local agencies and establish guidelines for local authorities to develop regulatory programs and minimum specifications.

3. Uniform citation and complaint. State and localities should develop methods of issuing citations for sanitary ordinance or code violations to expedite enforcement and permits.

4. Small claims courts. The less formal procedures of small claims courts should be used to accelerate the imposition of fines and forfeitures for violations of on-site system standards.

5. Civil service status. Regulatory officials should be given job security to protect them from pressures exerted by parties with vested interests.

Operation

1. Septic tank maintenance permit. Permits could be issued every 1 to 3 years upon receipt by the regulatory agency of proof that the septic tank has been inspected and/or pumped.

TABLE 25. ALTERNATIVES AND COMPLIMENTARY CONTROL METHODS FOR INSTALLATION OF ON-SITE DISPOSAL SYSTEMS (Stewart, 1975a)

Phases of Control	Soil and Site Requirements	System Design	System Construction
Pre-Approval Administrative Control Functions	(1) Agency has permit program requiring application (2) Knowledge of location by official (3) Use of soil maps etc. (4) On-site investigation by agency	(1) Require information on permit application (2) Check by agency of design (3) Alternatively, design done by agency	(1) Education Program promoting good construction and monitoring of system
Post-Approval Administrative Control Functions	(1) On-site inspection by official to verify the information (after the fact)	(1) On-site investigations during construction to verify design (not after the fact)	(1) On-site inspections (2) Certification that official inspections at critical times were made

2. Conditional sanitary permit. A permit to install and use a system could be issued which would become void upon proof by the regulatory authority that proper maintenance had not been performed.

3. Location and filing requirement. This would require the filing of an "as-built" plan showing the size and location of the disposal system to simplify servicing.

Failure

1. Sanitary surveys. Periodic inspections could be made to locate failing systems within the jurisdiction of the regulatory authority.

2. Violation as an encumbrance. Notice of a violation could be filed by the regulatory authority with the register of deeds to alert potential buyers of the system.

3. Pre-sale inspection. The regulatory authority could inspect the systems whenever the property is sold.

4. Abatement costs. Regulatory authorities should have the authority to enter upon private land to abate a failing system and charge the owner for the work.

MANAGEMENT BY GOVERNMENTAL OR QUASI-GOVERNMENTAL UNITS

Although state and local authorities may regulate the installation and possibly the use of individual on-site systems, owners usually are responsible for the operation, maintenance and repair or on-site disposal systems. An alternative to this is the use of governmental or quasi-governmental units to install, operate, maintain, repair, and perhaps even own on-site systems (Otis and Stewart, 1976; Stewart, 1975b). Appropriate governmental units might include special purpose districts such as sanitary districts and drainage districts or might involve a special function of existing governmental units such as towns or counties.

Regardless of the type or size of the governmental unit, advantages of this would be: 1) control over system siting and design, 2) strict supervision of construction, 3) proper inspection and maintenance, and 4) immediate replacement of failing systems (probably paid for out of a replacement fund). However, there might be economies of scale in larger jurisdiction units which might make county level units more preferable.

The governmental or quasi-governmental unit should have the following powers (Otis and Stewart, 1976; Stewart, 1975b). Some additional optional powers also have been included (89).

1. Authority to plan, design, construct, operate and maintain all types of on-site systems, and the optional authority to own, purchase, lease and rent both real and personal property.

2. Authority to accept and utilize state and federal grants, and further sufficient authority to meet eligibility requirements imposed upon grant applicants.

3. Authority to contract, undertake debt obligations and to sue and to be sued.

4. Authority to raise revenue by fixing and collecting user charges for sewer services and as an optional revenue raising power, authority to determine and assess the benefit to any property served and collect that assessment from each property owner and/or have the power to levy a tax upon the landowners served.

5. Authority to plan and control how and when wastewater facilities will be provided to those within its jurisdiction, and

6. Authority to adopt needed rules and regulations governing the control and use of on-site systems within its jurisdiction.

Entities other than governmental or quasi-governmental units with powers to adequately manage on-site systems have been identified (Commission on Rural Water, 1974; Otis and Stewart, 1976; Stewart, 1975b): 1) incorporated cities and villages (local units of general government which have home rule powers), 2) counties and townships (local units of general government), 3) special purpose districts (quasi-governmental units), 4) private non-profit corporations, 5) rural electric cooperatives (cooperatives established to work with U.S.D.I.'s Rural Electrification Administration, 6) private businesses, and 7) other governmental agencies.

The use of governmental or quasi-governmental units to own and operate individual on-site disposal systems is a relatively new concept, but some have been successfully established in California, West Virginia, Florida and Wisconsin (Otis and Stewart, 1976). Experience seems to indicate that the concept is sound.

LAND USE IMPLICATIONS OF IMPROVED ON-SITE DISPOSAL TECHNOLOGY

The introduction of new on-site waste disposal technology raises questions about land use and land development. Regional planning commissions should understand the new technology and realize its possible impact on land use. Home and commercial development, as well as land values, may be increased, especially in areas where site conditions were hitherto unsuitable for on-site disposal systems. Planning commissions will be forced to redefine criteria for land use decisions.

Potential Areas of Impact on Land Use

In many parts of this country, planners have relied on the unsuitability of lands for conventional systems as a tool in determining land use. Central public sewerage is often too expensive for sparsely populated areas. Thus, development is effectively prevented in rural areas unsuited for septic tank systems by what is commonly referred to as *de facto* zoning. The availability of alternate systems for on-site waste disposal or more cost effective methods of public sewerage could have impact on land use, especially *de facto* zoning.

Alternate Systems - Case Studies of Potential Impact

Two case studies (Amato and Goehring, 1974; Water Resources Management Workshop, 1973) suggest that alternate on-site disposal systems could have considerable impact, especially in these areas which currently rely on *de facto* zoning. Any area meeting the reduced site and soil requirements of alternative systems will be open to development unless curtailed by other mechanisms of land use control. The development of resource data to advise officials of appropriate governmental units should be a top priority assignment of planning commission (Amato and Goehring, 1974).

Soil Surveys to Predict Land Use Implications

Soil surveys have been used, in part, to evaluate potential sites for wastewater disposal with conventional septic tank systems. Soil surveys now should be also used to predict the potential of sites for alternate methods of waste disposal (Beatty and Bouma, 1973). Such information is vitally needed in jurisdictions which rely on *de facto* zoning instead of more valid methods of land use controls. Land use controls should be based on the desires of the community and not fortuitous circumstance that certain lands are not suited for conventional septic systems.

SECTION 9

ALTERNATIVE SELECTION

THE SELECTION PROCESS

The choices available for the individual home wastewater disposal are numerous, yet only a selected few will prove to be both economically and environmentally acceptable. The selection process involves the evaluation of technical feasibility, cost effectiveness and administrative feasibility. All three are extremely important to the successful execution of the project.

Table 26 is a listing of some of the on-site wastewater disposal options available. It is first necessary to evaluate the design constraints for the site, including soils, topography, geological characteristics, climate and water quality requirements. Once these physical constraints have been delineated, an orderly selection of options may be undertaken. With each of the feasible alternatives, an appropriate institutional framework must be developed to insure appropriate construction, operation and maintenance of the system. Finally, the capital and operation and maintenance costs of each feasible alternative, including cost of the administrative framework must be determined. This is necessary to select the most cost effective alternative from those evaluated.

Examples of some potential system flowsheets are depicted in Table 27. It should be emphasized that although technical feasibility of many of these systems is proven, extensive field testing to determine process reliability and effectiveness of institutional controls still needs to be accomplished.

COSTS OF TREATMENT AND DISPOSAL SYSTEMS

With the exception of standard septic tank-soil absorption systems, the cost of alternative on-site treatment and disposal systems are not well documented, owing to the lack of a large enough data base. Estimates of unit cost ranges are presented in Table 28, and are based primarily upon Wisconsin experiences. It should be recognized that these costs are merely estimates based on relatively sparse experience and individual systems should be evaluated on a site by site basis. In all instances, in-house devices may substantially decrease overall costs for treatment and disposal.

TABLE 26. ON-SITE WASTEWATER TREATMENT AND DISPOSAL COMPONENTS

- In-House
  - Water Conservation
    - Flow Control
    - Reuse and Recycle
  - Waste Segregation
    - Non-Water Carriage Toilets
      - Chemical
      - Biological
      - Recycle
      - Incinerator
    - Very-Low-Flow Toilets
  - Household Product Selection
  - Household Appliance Selection

- Anaerobic Processes
  - Septic Tanks
  - Fixed Media Filters
    - Sand
    - Synthetic Media

- Aerobic Processes
  - Suspended Growth
    - Activated Sludge
    - Lagoons
  - Fixed Media
    - Soil Mantle
    - Granular Filters
    - Coarse Media
      - Rotating Biological Contactors
      - Trickling Filters
    - Emergent Vegetation

- Physical Chemical Processes
  - Ion Exchange
  - Chemical Precipitation
  - Disinfection
    - Halogens
    - Ultraviolet
    - Ozone
  - Adsorption

- Land Application for Disposal
  - Soil Absorption
  - Mounds
  - Irrigation
  - Lagoons (Absorption)
  - Evapotranspiration

TABLE 27. POSSIBLE ON-SITE FLOW SHEETS UNDER SELECTED CONSTRAINTS

Constraints	Process Flow Sheet	Cost	Maintenance	Reliability
Good Soil and Site Conditions	Septic Tank-Soil Absorption	Low	Low	Excellent; if proper construction, maintenance.
High bedrock	Septic Tank-Mound- Soil	High	Low	Excellent if proper construction, maintenance.
	or			
	Water Conservation- Septic Tank-Sand Filtration-Chlorination- Surface Water	High	Moderate	
Poor Soil/Topography Surface Water: BOD-30 mg/L SS-30 mg/L E Coli < 200/100	Water Conservation-Activated Sludge-Sand Filter-UV	High	High	Good, if proper construction, maintenance.
	or			
	Waste Segregation-Black Water Holding; Grey water: Septic Tank-Sand Filter	High	Moderate	Need Data.
Poor Soil/Topography Lake: BOD-10 mg/L SS-10 mg/L N < 1 mg/L P < 1 mg/L E Coli < 100/100	Water Conservation-Septic Tank-Sand Filter-Denitrification-P Absorption-Iodine	High	High	Need Data.
	or			
	Waste Segregation-Biological Toilet; Grey Water: Septic Tank-Sand Filter-NH_4^+ Ion Exchange- P Adsorption	High	High	Need Data.

TABLE 28. CAPITAL COSTS AND OPERATION AND MAINTENANCE COSTS

Septic Tank (1,000 gal):	
Installed Cost	350 to 450
Operation Cost Per Year	10
Aerobic:	
Installed Cost	1,500 to 2,500
Maintenance Per Year	65 to 110
Power Per Year, 2.4 to 7.4 kwh/d @ 4¢/kwh)	35 to 60
Sand Filter:	
Installed Cost	15 to 20/ft^2
Maintenance Per Year	1 ft^2
Pump Chamber:	
Installed Cost:	
Chamber	200 to 250
1/2-hp Sump Pump and Controls	300 to 350
Operation Cost[a] Per Year, 0.1 kwh/d @ 4¢ per kwh	2
Chlorine and Settling Chamber:	
Installed Cost	600 to 800
Operation Cost[1] Per Year (Chemical)	26
Ultraviolet Irradiation:	
Installed Cost	1,100 to 1,500
Power, 1 1/2 kwh/d @ 4¢ per kwh	20
Maintenance Per Year	40 to 60
Soil Absorption System:	
Installed Cost	1 to 1.25/ft^2
Maintenance Per Year	-
Mound:	
Installed Cost	2,500 to 4,000
Maintenance Per Year	18 to 25
Power Per Year	2
Evapotranspiration:	
Installed Cost	.75 to 1.00 ft^2
Maintenance Per Year	-

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[1] Does not include pump replacement.

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APPENDIX A

WASTEWATER CHARACTERISTICS AND TREATMENT

PART 1

THE CHARACTERIZATION OF SMALL WASTE FLOWS

INTRODUCTION

The characteristics of waste flows from residential dwellings, as well as non-residential establishments can have a profound effect on the performance of individual treatment and final disposal methods. Various water-use events within a dwelling or establishment create an intermittent flow of wastewater which can vary widely in strength and volume. Quantitative and qualitative characterization information is necessary: (1) to provide for the effective design of treatment and disposal systems, (2) to facilitate the development of methods to beneficially alter the typical waste characteristics, and (3) to facilitate the development of methods to recycle resources in a beneficial manner. To enhance the existing characterization data base, a major study was conducted.

This characterization study was accomplished in several phases. The first phase included the characterization of rural household wastewater. A major effort was expended in this phase since it was felt to be of prime importance. Three objectives were established for this effort: (1) identify the contributions made to the wastewater stream by various individual water-using events within the home, (2) identify the patterns in water usage and hence wastewater production on an hourly, daily and seasonal basis, and (3) identify the qualitative characteristics of the wastewater resulting from the various water-use events.

An attempt was made to obtain a sufficient number of sites to yield a variety of family sizes and types. At each study home, efforts were made to monitor water use and/or wastewater production for a sufficient length of time to provide representative, useful characterization information. Water use (wastewater production) was monitored at eleven homes for a total of 434 days. Chemical/physical wastewater characteristics were identified through monitoring at four households for a total of 35 days. The microbiological characteristics of selected household event wastewaters were determined through in-house sampling at each of six households.

The second phase of this study involved the characterization of wastewaters produced by various commercial establishments. A major objective of this phase of the study was to compile a comprehensive summary of the existing characterization data which heretofore had been scattered in bits and pieces amidst the literature and in other dark places. A second objective was to consider and evaluate various methodologies used for estimating water use/

wastewater production at facilities serving a transient population. Finally, the existing characterization data base was to be expanded where necessary and feasible.

The final phase of this study involved an investigation of methods for in-house alteration of the typical characteristics of small wastewater flows, primarily those from households. Although it was not among the major objectives of this study to actively evaluate methods of in-house alteration, since the results of the characterization phases identified areas offering potential for wastewater alteration, an investigation and discussion of this topic was deemed necessary.

CHARACTERISTICS OF HOUSEHOLD WASTEWATER

The first phase of the characterization effort involved the quantitative and qualitative characterization of rural household wastewater. The available characterization information was reviewed, compiled and summarized prior to conducting field studies.

Review of the Existing Literature

A limited number of investigations have been conducted to delineate the characteristics of the water usage and wastewater production of individual household water-using events and activities. A brief discussion of these investigations follows. In 1968, Olsson, Karlgren and Tullander (1968) reported the results of a study investigating the quantities and types of pollutants in several fractions of domestic wastewater. The investigation was conducted at a building containing 25 apartments situated in a suburb of Stockholm, Sweden. The drainage system for the building was installed so that wastewater could be obtained separately from each of the following groups: 10 kitchens, 10 bathrooms (excluding toilet wastes), 15 kitchens and bathrooms combined (excluding toilet wastes), a laundry and 28 vacuum toilets. The wastewater flow volumes were measured and samples of each group were analyzed for a variety of parameters during a 12-week period (January 11 to April 4, 1965).

The waste flow volumes were measured continuously throughout the study for each category of wastewater. The samples used for qualitative analyses were either random grab or flow composited samples on a daily or weekly basis, depending on the parameter of interest and the category of wastewater. Basically, BOD_5 analyses were performed on daily composites; the pH, temperature and specific conductance were measured continuously; and the remaining chemical/physical parameters were based on weekly composites. In addition, during week 3 the daily variations for all parameters were studied. Throughout the study, however, samples of the toilet and laundry waste were taken randomly during each week. Bacteriological analyses were conducted on the wastewater from kitchens and bathrooms during week 3 for total bacteria, total coliform and fecal coliforms. During week 12 similar analyses were performed on the wastewater from the combination kitchen-bathroom group and the toilet wastes.

To express the results in terms of mass/capita/day, interviews were conducted with the occupants of each apartment to determine the total number, occupation, sex, age and habits of the residents. One interview was held immediately before the study and a second immediately after. A summary of selected mean results determined in the study is shown in Table A-1.

TABLE A-1. SELECTED CHARACTERISTICS OF VARIOUS COMPONENTS OF RESIDENTIAL WASTEWATER (Olsson, Karlgren and Tullander, 1968)

Parameter	Kitchens		Bathrooms		Combined Kitchens & Bathrooms	Laundry	Vacuum Toilet
	Week 1-12	Week 3	Week 1-12	Week 3	Week 3	Week 1-12	Week 1-12
BOD_5, g/cap/day	17	-	5	4	20	3	20
Total P, g/cap/day	0.3	0.3	0.6	0.6	0.9	1.3	1.6
PO_4-P, g/cap/day	0.01	0.01	0.01	0.01	0.02	-	-
Kjeldahl N, g/cap/day	0.6	0.6	0.3	0.3	0.9	0.2	11
NH_4-N, g/cap/day	0.04	0.04	0.03	0.011	0.051	-	-
NO_2-N, g/cap/day	0.001	0.001	0.001	0.001	0.002	-	-
TS, g/cap/day	36	31	22	19	50	19	53
TVS, g/cap/day	27	23	10	9	32	7	39
TSS, g/cap/day	13	11	3	3	14	2	30
TVSS, g/cap/day	12	10	2	2	12	1	25
pH	7.3	7.2	8.1	7.8	-	9.8	8.9
Temp, °C	28	28	25	25	26	-	-
Plate Count-35°C, Log no./cap/day	-	10.55	-	10.61	-	-	10.79
Coli - 35°C, Log no./cap/day	-	10.08	-	9.0	-	-	9.68
Coli - 44°C, Log no./cap/day	-	9.36	-	8.30	-	-	9.58
Flow, L/cap/day	51	44	62	55	99	8.5	8.5

As part of a study to evaluate the feasibility of various household waste-flow reduction techniques, information was obtained on residential water use/wastewater flow volumes by Cohen and Wallman (1974). Eight families were included in the study, six in southeastern Connecticut and two in San Diego, California. At each of the homes, water meters were installed on the house supply line and the individual lines to the toilet and bath/shower. At five of the homes the supply line to the laundry was metered also. Water use was recorded weekly by the homeowners over two, six-month periods, before and after waste-flow reduction devices were employed. The results of this monitoring are summarized in Table A-2.

TABLE A-2. HOUSEHOLD WATER USE - L/cap/day
(Cohen and Wallman, 1974)

Home	Inlet	Toilet	Bath/ Shower	Laundry	Kitchen & Bathroom
1	145	34.4	14.0	69.0	
2	384	104	36.0	34.8	
3	157	41.0	10.8	-	
4	220	72.5	16.6	-	
5	263	113	40.2	-	
6	190	44.8	28.4	37.8	
7	143	42.1	24.9	31.4	
8	179	69.8	20.8	26.5	
Average	210	65.1	23.8	39.8	68.3
% of Inlet	-	30.6	11.6	19.1	29.4

Hot and cold water use by the bath and laundry were also recorded as part of this study. The average ratio of hot to cold water use was calculated to equal approximately 1.2 for the bath and 0.8 for the laundry.

A third characterization study was performed by Laak (1975) in 1972 at the University of Connecticut involving five families. The first phase of the study identified the volume of wastewater generated by various water-using events. For the kitchen sink, bathroom sink and bathtub, relationships between depth and volume were obtained at each home. Measurements of the liquid depth before draining the wastewater from each of these fixtures were recorded by the user. For clothes washing, the average quantities of water used for each machine setting were used to estimate wastewater production. For toilet flushing, the volume of water used per flush was calculated from the flush tank dimensions. The number of flushes per day were recorded by counters activated by the flush lever which enabled the volume of water used per capita per day to be calculated. The results of this characterization are given in Table A-3.

TABLE A-3. MEAN WATER CONSUMPTION BY HOUSEHOLD EVENTS - L/cap/day (Laak, 1975)

Home	Kitchen Sink	Bath	Bathroom Sink	Laundry	Toilet	Total
A	12.1	58.2	11.3	54	112	247
B	34.4	20.0	12.1	7.9	138	212
C	12.9	22.3	5.7	16.3	42.3	99
D	7.9	18.9	3.8	29.9	51.8	112
E	7.9	37.8	10.2	17.0	49.9	123
Weighted Average	13.6	32.1	7.9	28.0	74.8	156

The second phase of this study involved identifying the quality of the wastewater generated from the plumbing fixtures. At each of the five residences, the wastewater was sampled immediately after dishwashing, bathing, laundering, hand or face washing and tooth brushing. Analyses were conducted for a variety of chemical/physical parameters. The pollutant concentrations determined were converted to mass/capita/day values by utilizing the mean flow results shown in Table A-3. The wastewater from toilets was not sampled. Instead, fresh feces from two persons and urine from student washrooms were collected on 12 separate occasions. These washroom samples were prepared for analysis by dissolving 0.1 gram of wet feces or 1 milliliter of urine in one liter of distilled water. The average discharge of urine was estimated at 1200 mL/day per adult and 800 mL/day per child; wet fecal output was estimated at 130 gram/day per adult and 90 gram/day per child. The weighted mean production rates for the 10 adults and 6 children of this study were calculated to be 115 gram feces/day and 1050 mL urine/day. Using these production rates and the results of the sample analyses, the average fecal and urine contributions in toilet wastewater were calculated. The individual event pollutant contributions determined in this study are summarized in Table A-4.

Laak also investigated the pollutant load contributed by the manufactured materials or consumer products used at each plumbing fixture. To accomplish this, the brands of materials used in each family were independently purchased and analyzed.

In 1974, the results of a study to determine the character of individual household wastewaters were reported by Ligman, Hutzler and Boyle (1974). The first phase of the study involved a wastewater generation survey to determine the number of uses per person per day for each of five household events: toilet, bath/shower, laundry, dishwashing and garbage disposal use. Questionnaires were completed by the residents of 20 rural homes, four urban homes and six apartment households. The residents recorded the daily number of occurrences for the five household events over a one-week period. The results of the survey for rural homes are shown in Table A-5. Urban event occurrences

TABLE A-4. AVERAGE POLLUTANT CONTRIBUTIONS FROM INDIVIDUAL HOUSEHOLD EVENTS - grams/cap/day (Laak, 1975)

Parameter	Kitchen Sink	Bathing	Bathroom Sink	Clothes Washing	Toilet	Total
BOD_5	9.20	6.18	1.86	7.90	23.54	48.69
COD	18.80	9.08	3.25	20.30	67.78	119.4
Total Kjeldahl Nitrogen	*	-	-	-	-	-
NH_3-N	0.074	0.043	0.009	0.316	2.78	3.22
NO_3-N	0.0076	0.0116	0.0022	0.0353	0.016	.0727
PO_4-P	0.173	0.030	0.386	4.79	6.47	11.86

* Samples from Homes A, B, and E were analyzed, yielding a range of values equal to 0.054 to 0.410 grams/cap/day.

TABLE A-5. EVENT USAGE SURVEY RESULTS AT RURAL HOMES (Ligman, Hutzler and Boyle, 1974)

	Toilet Use	Bathing	Clothes Washing	Manual Dishwashing	Automatic Dishwashing	Garbage Disposal
Statistic	uses/cap/day			uses/home/day		
Mean	3.6	0.44	0.34	1.9	1.3	2.9
Responses	62	33	27	26	9	7
Range	1.5-7.2	0.14-1.03	0.07-0.57	0.5-3.7	0.4-2.5	1.1-5.0
95% Confidence Interval	3.0-4.5	0.36-0.53	0.26-0.41	1.6-2.3	0.9-1.8	1.7-3.6

did not differ significantly from those shown for rural homes. To estimate water use/wastewater production, the survey responses (number/capita/day) were multiplied by typical water use values given in the literature (volume/occurrence). The results calculated have been summarized in Table A-6.

The second phase of this study involved sampling bath, shower, dishwashing and laundry wastewaters at each of ten rural households and five apartment households (January to May of 1972). Three samples of each event were taken by the occupants of each home. If a family used an automatic dishwasher, they thoroughly rinsed the dishes before inserting them into the dishwasher, sampled the rinse water and noted its volume. For automatic clotheswashers samples were only taken from the first discharge from the machine.

TABLE A-6. CALCULATED RURAL WASTEWATER FLOW VOLUMES* - L/cap/day (Ligman, Hutzler and Boyle, 1974)

Day of Week	Mean	Range	95% Confidence Interval
Monday	187	76 - 415	140 - 235
Tuesday	170	73 - 363	130 - 209
Wednesday	144	73 - 239	119 - 167
Thursday	167	57 - 398	129 - 204
Friday	153	55.6 - 465	118 - 287
Saturday	190	60.5 - 614	128 - 252
Sunday	165	31 - 346	122 - 208
Average	168	31 - 614	153 - 183

* Based on the recorded occurrences per capita per day and the following waste flow volumes:

Toilet	-	18.9 L/use (5 gal/use)
Bath/Shower	-	94.5 L/use (25 gal/use)
Clotheswasher	-	189 L/use (50 gal/use)
Dishwashing	-	26.5 L/use (7 gal/use)
Garbage Disposal	-	7.6 L/use (2 gal/use)

The average pollutant concentrations for the wastewater from the entire clotheswasher cycle were estimated to be 58% of the concentration in the first discharge. For the bath, shower and dishwashing wastewaters, the occupants were asked to estimate the volume of wastewater from which they obtained each sample, while clotheswasher discharges were assigned an arbitrary value of 151 L (40 gal). The estimated wastewater volumes were combined with the measured concentrations to determine mass loadings per event (Table A-7).

In addition to the above studies, an extensive literature review was conducted. Based on the results of both, the mean characteristics of individual household wastewaters were estimated as shown in Tables A-8 and A-9.

In 1975, Bennett and Linstedt (1975) reported the results of a study conducted by the University of Colorado to determine the characteristics of household wastewater and evaluate the effect of these characteristics on various types of treatment processes. The wastewater characterization phase involved water-use monitoring and wastewater sampling of the following household events: sink use, toilet flushing, garbage grinding, bathing, clotheswashing, and dishwashing.

Water-use monitoring was conducted at each of five homes in the Boulder, Colorado area for a one- to two-week period. A recording device was attached to the water meter at each study home which provided a record of water use versus time of day. To identify which household water-use event generated each recorded volume, the residents noted the type of event, time of occur-

TABLE A-7. SELECTED CHARACTERISTICS OF INDIVIDUAL EVENT WASTEWATER - grams/event (Ligman, Hutzler and Boyle, 1974)

Parameter	Clothes Washing	Bathing	Dishwashing
BOD_5	38.6* 4.5 - 112	17.7 10.0 - 28.6	11.8 1.8 - 27.2
Total Solids	160 53.1 - 335	42.2 22.2 - 72.6	20.0 6.8 - 45.4
Total Suspended Solids	29.5 0.45 - 79.4	11.4 1.8 - 37.7	5.9 0.90 - 16.8
Total Volatile Solids	57.7 30.0 - 96.2	15.0 7.3 - 25.9	14.5 3.2 - 38.1
Total Volatile Suspended Solids	16.3 0.45 - 47.7	2.3 0.45 - 7.7	5.0 0.45 - 13.6
Number of Samples	8 - 24	8 - 13	23 - 33

* Mean over range

TABLE A-8. INDIVIDUAL EVENT WASTEWATER QUALITY SUMMARY - grams/event (Ligman, Hutzler and Boyle, 1974)

Parameter	Clothes Washing	Bathing	Dishwashing	Garbage Disposal
BOD_5	38.6* 4.5 - 112	17.7 10.0 - 28.6	11.8 1.82 - 27.2	41.3 33.1 - 49.5
Total Solids	160 53.1 - 335	42.2 22.2 - 72.6	20.0 6.8 - 45.4	81.7 75.4 - 90.8
Suspended Solids	29.5 0.45 - 79.4	11.4 1.82 - 37.7	5.9 0.91 - 16.8	58.1 49.9 - 66.7
Fats	-	-	-†	10.4 5.9 - 15.0
Total Nitrogen	-	-	-†	1.2 -
Total Phosphorus	9.1 3.6 - 12.7	-	1.36 Trace - 2.72	-
Flow, Liters	151 128 - 223	94.5 37.8 - 113	- 18.9 - 60.5	5.7 3.8 - 7.6

* Mean value over range of values

† Values not identified

TABLE A-9. INDIVIDUAL EVENT POLLUTANT CONTRIBUTIONS SUMMARY - gram/cap/day (Ligman, Hutzler and Boyle, 1974)

Parameter	Clothes Washing	Bathing	Dishwashing	Garbage Disposal	Toilet
BOD_5	9.5* 1.4 - 28.1	9.1 5.0 - 14.5	5.9 0.45 - 6.81	30.9 25.0 - 37.2	23.6 17.3 - 37.7
Total Solids	40.0 13.2 - 84.0	20.9 11.4 - 36.3	10.0 1.8 - 11.4	61.3 56.8 - 68.1	97.2 82.6 - 125.3
Suspended Solids	7.26 0.45 - 20.0	5.45 0.91 - 19.1	2.72 0.45 - 4.09	43.6 37.7 - 49.9	30.9 22.7 - 46.3
Fats	-	-	-†	7.72 4.54 - 11.4	4.54 0.91 - 8.17
Total Nitrogen	-	-	-†	0.91 -	16.8 1.36 - 22.7
Total Phosphorus	2.27 0.91 - 3.18	-	0.45 Trace - 0.91	-	1.36 1.36 - 3.63

* Mean value over range of values

† Values not identified

rence and user age group on data sheets provided. The results obtained from this monitoring are summarized in Tables A-10 and A-11.

TABLE A-10. INDIVIDUAL EVENT WATER USE (Bennett and Linstedt, 1975)

Event	L/use	uses/cap/day	L/cap/day
Toilet	15.5	3.6	55.6
Bathroom Sink	7.6	2.5	19.0
Bathing	103	0.32	32.9
Kitchen Sink	3.8	2.60	9.8
Garbage Disposal	7.9	0.40	3.0
Automatic Dishwasher	26.5	0.15	4.2
Clotheswasher	43.8	0.30	43.8
Total	-	-	168

TABLE A-11. RESIDENTIAL WATER USE - L/cap/day
(After Bennett and Linstedt, 1975)

Home	Sink	Toilet	Garbage Disposal	Bathing	Clothes-washer	Dish-washer
1	36.1	68.7	2.4	13.1	55.4	4.6
2	17.4	38.8	0.4	31.8	26.8	1.3
3	45.7	89.9	4.0	61.7	20.9	7.3
4	76.7	108.4	16.3	29.6	59.8	-
5	17.4	37.8	1.7	40.9	34.3	3.7

The flow data was also analyzed to provide hourly water use patterns. To illustrate the daily variations, water use versus time of day curves were drawn. The summary curve for all five sites is shown in Figure A-1.

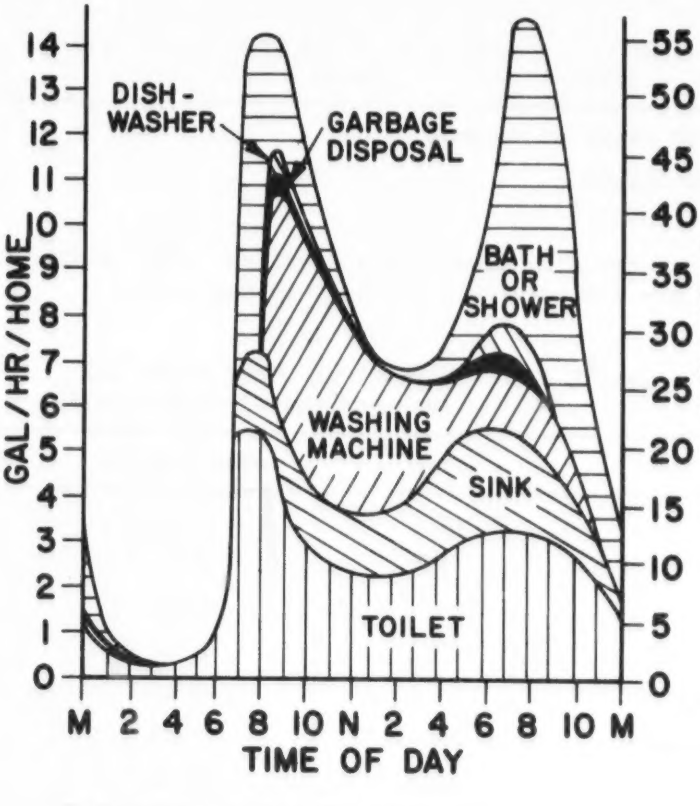

Figure A-1. Daily water-use patterns (Bennett and Linstedt, 1975).

The second phase of the study determined the qualitative characteristics of the wastewaters produced by each of the six household events studied in the water-use phase. Grab samples were obtained from the wastewater produced by each event (excluding the toilet) at each of the original five homes, plus two additional homes. Individual samples of feces, urine, and toilet paper were analyzed and the average strength of the toilet wastes was estimated. Utilizing the mean flow results determined in the water-use monitoring phase, the pollutant concentrations were converted to mass/capita/day values as shown in Table A-12.

To determine the variation in pollutant contribution throughout the day, the hourly water-use curve (Figure A-1) was combined with the wastewater quality data (Table A-12). Curves such as that shown in Figure A-2 for the hourly COD profile were developed.

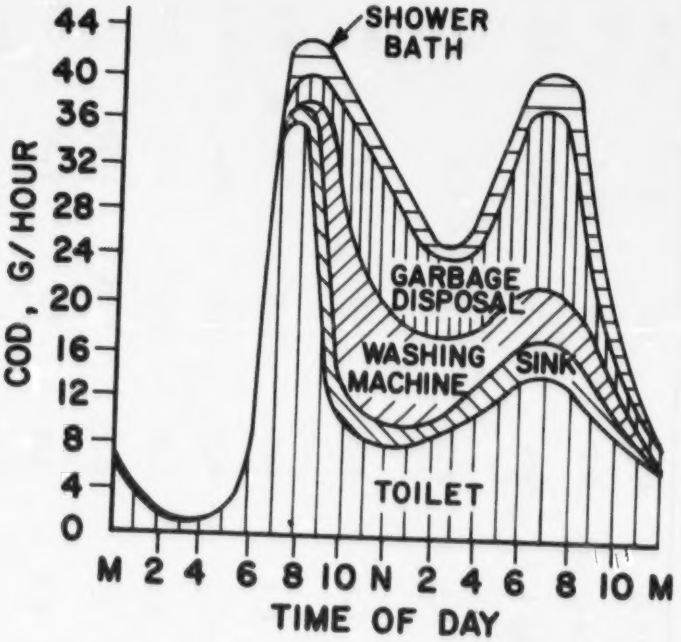

Figure A-2. Hourly COD profile (Bennett and Linstedt, 1975).

As part of this investigation, the pollutant contributions of several cleaning products commonly used in the home were also identified.

TABLE A-12. POLLUTANT CONTRIBUTIONS FROM INDIVIDUAL HOUSEHOLD EVENTS (Bennett and Linstedt, 1975)

Parameter	Kitchen Sink	Bathroom Sink	Bath-Shower	Clothes-washer	Dish-washer	Garbage Disposal	Toilet	Total
BOD_5, g/cap/day	10.6	4.9	3.2	8.7	0.5	12.3	6.9	47.1
COD, g/cap/day	16.2	9.2	7.2	24.0	0.9	35.6	65.0	157
Total Solids, g/cap/day	13.4	9.4	10.9	33.3	3.8	32.5	76.5	178.8
Total Volatile Solids, g/cap/day	9.5	2.2	0.6	5.6	0.6	22.1	56.7	97.3
Suspended Solids, g/cap/day	2.1	4.3	0.9	3.4	0.1	20.2	36.5	67.5
Volatile Suspended Solids, g/cap/day	2.1	4.3	0.9	3.2	0.1	19.2	30.7	60.5
Total Nitrogen, g/cap/day	1.1	0.0	0.0	0.2	0.0	0.2	5.2	7.4
Phosphate-P, g/cap/day	0.0	0.0	0.0	0.6	0.0	0.1	3.1	3.8
MBAS, g/cap/day	-	0.11	1.74	1.53	-	-	-	3.38
pH	6.6	7.9	8.2	7.8	7.6	6.4	5.6	7.4
Temp, °C	43	20	38	53	62	30	21	36
Flow, L/cap/day	9.8	18.9	32.9	43.8	4.2	3.0	55.6	168

Data Generation Methods

Field studies to further delineate household wastewater characteristics were accomplished in three phases: (1) water use characterization (wastewater production), (2) wastewater quality characterization for chemical/physical parameters, and (3) wastewater quality characterization for microbiological parameters. A discussion of the methods used for each of these phases follows. It should be noted that certain of the information presented has been discussed previously (Siegrist, 1975; Siegrist, Witt and Boyle, 1976; Witt, 1974a; Witt, 1974b).

Water Use Characterization--

Eleven rural homes were monitored offering a variety of family types and sizes as described in Table A-13. Five water-use events were included: toilet usage, bathing, clotheswashing, dishwashing and water softening.

TABLE A-13. FAMILY INFORMATION - WATER USE MONITORING PHASE

Family Unit	Adults	Children (Age)	Bathrooms	Automatic Clothes Washer	Automatic Dish Washer	Water Softener	Occupation of Head of Household
A	2	2 (8,18)	2 1/2	Yes	Yes	Yes	Herdsman
B	2	1 (15)	1 1/2	No	Yes	No	Earth Contractor
C	2	2 (3,5)	1	Yes	No	Yes	Herdsman
D	2	4 (10,12, 17,19)	2	Yes	Yes	Yes	Resort Employee
E	2	1 (9 mo.)	2	Yes	Yes	No	Pharmacist
F	2	3 (6,8,9)	1 1/2	Yes	No	No	Paper Mill Worker
G	2	5 (4,9,15, 17,18)	1 1/2	Yes	No	Yes	Dairy Farmer
H	3	0	1	Yes	No	Yes	Farm Worker
I	2	3 (2,3,5)	1 1/2	Yes	Yes	No	Meat Cutter
J	2	5 (3,7,11, 16,17)	1 1/2	Yes	No	No	Agronomist
K	2	2 (8,15)	2	Yes	No	Yes	Agriculture Professor

Monitoring equipment--When selecting the water-use monitoring equipment, a prime consideration was to avoid interruption of the normal activities within the study homes. As a result, a chart recorder (F.S. Brainard and Company, New Hope, PA, Model 67 RF) driven by a household water meter was chosen (Figure A-3).

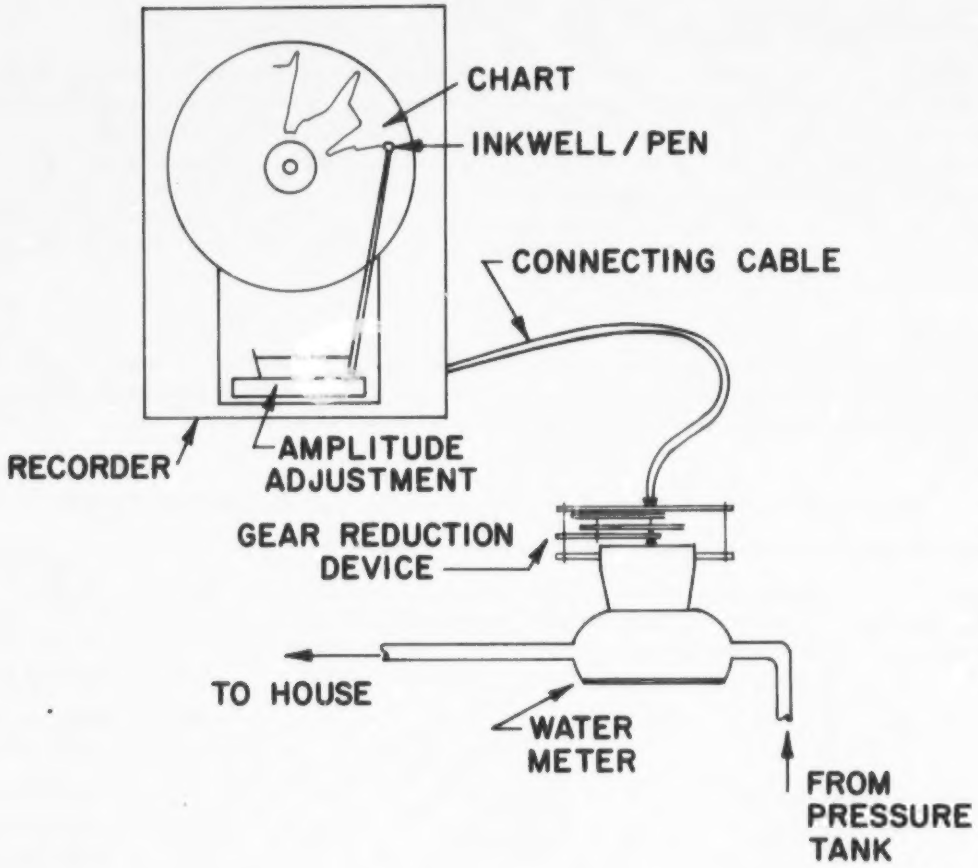

Figure A-3. Water use monitoring equipment.

The recorder employed a 20 cm (8 in) diameter metal disc to which calibrated paper charts could be attached. The disc and attached chart were rotated by an eight-day, spring-wound clock at a rate such that charts covering one-week periods could be utilized. The recorder was connected mechanically to a water meter located in the water supply line at the outlet of the household pressure tank. As water was used, the pointer on the face of the water meter caused an ink pen on the recorder to be driven back and forth across the paper chart. The volume of water used was directly proportional to the distance the pen moved across the chart, and the time of use was indicated by time calibrations on the chart.

Data collection--During operation at each of the study homes, the recorder charts were changed on a weekly basis. This eliminated much of the homeowner's involvement and lessened the chance of altering the normal life style of the family. To expedite data collection at the eleven homes, several chart recorders were utilized simultaneously. The recorders were moved between the eleven homes during the study, with efforts made to obtain four weeks or more of data at each, in segments no smaller than seven continuous days. Further, to allow winter to summer water-use comparison, at least five weeks of summer data and four weeks of winter data were obtained at each of three households. In total, data were collected for 434 days.

Data interpretation and analysis--Each recorder chart generated was analyzed to determine the type of water-using event, the time of occurrence and the volume of water used for each of the many water-use patterns recorded. To aid in event identification, background information on family habits and appliances within the home was obtained using a short questionnaire. In addition, a check sheet was used during a two- to three-day period during the initial installation of the monitoring equipment at each home. The residents recorded the time of occurrence for each type of event three times. This produced example chart patterns for each event. Utilizing this background information, individual water-using events were identified on each chart by comparing the flow pattern recorded with the characteristics of the example events. The volume of water used by each identified event was determined by measuring the distance the ink pen moved across the calibrated paper charts.

As a check on the chart record and interpretation, the household water meter reading was noted at the beginning and end of each recording period. This total water use figure was used as a check against the total volume determined by summing up the volumes measured for the individual events. Additional checks included the installation of counters on the toilets, consultation with the homeowner regarding any extraordinary flows, and cross-checking the data where possible with the qualitative characterization phase of this study.

Two computer programs were developed to compile the data collected. The first program analyzed the data by hour of the day and type of event for each of the study homes individually and all homes combined. The percentage of the hourly flow, the average event flow per capita per hour and the standard deviation of the event flow were computed. This program also provided a summary of an entire 24-hour day including the number of event occurrences per capita, the average and standard deviation of the event size, the average flow per capita, the percentage of the total daily flow and the number of times the event was observed during the sampling period.

The second computer program processed the water-use data in a similar manner, but by day of the week rather than hour of the day. The program computed the number of event occurrences per capita, the mean and 90 percent confidence interval for the event flow volume per capita, and the mean percent of total daily flow.

Wastewater Quality Characterization: Chemical/Physical--

Wastewater quality studies for chemical/physical parameters were conducted at three of the residences studied in the water use (wastewater production) phase. Since one of the three residences was occupied by a second family during the course of the wastewater quality study, it was treated as two residences. Thus, four families were involved in the study as described in Table A-14.

TABLE A-14. FAMILY INFORMATION - WASTEWATER QUALITY PHASE

Family Unit	Adults	Children (Age)	Bathrooms	Automatic Clothes Washer	Automatic Dish Washer	Garbage Disposal	Occupation of Head of Household
C	2	2 (3,5)	1	Yes	No	Yes	Herdsman
G	2	5 (4,9,15, 17,18)	1 1/2	Yes	Yes	No	Dairy Farmer
I	2	3 (2,3,5)	1 1/2	Yes	Yes	No	Meat Cutter
C′	2	1 (1)	1	Yes	No	Yes	Herdsman

The wastewaters generated through occurrence of the following water-using events were selected for this qualitative characterization: toilet flushing, (nonfecal and fecal), garbage grinding, kitchen sink usage, automatic dishwashing, clotheswashing (wash and rinse cycles) and bathing.

Monitoring equipment--At the onset, considerable effort was expended in selecting the monitoring equipment. As in the water-use monitoring phase, a primary consideration was to minimize the homeowner's involvement in the study. Further, the equipment selected had to provide for the collection of samples from individual event wastewaters, many of which were very heterogeneous. After review of commercially available products indicated the lack of a suitable sampler, a sampling system was designed and constructed by project personnel (Siegrist, 1975). As shown in Figure A-4, the sampler consisted basically of three modules with many separable parts to facilitate a portable system. Module 1 consisted of a polyethylene receiving tank, a grinder pump (Model SPG-150 M made by Hydromatic Pump Company of Hayesville, Ohio) and certain ancillary equipment. The polyethylene tank was 46 cm (18 in) in diameter and 81 cm (32 in) deep. A 5 cm (2 in) diameter section of plastic pipe connected the center of the tank bottom to the intake of the grinder pump. To the outlet of the pump was connected a 76 cm (30 in) section of 3.2 cm (1.25 in) diameter plastic pipe terminating in a cross fitting attached to a recycle, sample, and discharge line. The recycle line, 1.9 cm (0.75 in) diameter plastic pipe, directed effluent from the grinder pump back to the receiving tank. The sample line, 1.3 cm (0.50 in) diameter rubber tubing, connected a solenoid valve attached to the cross with Module 3. The discharge line, 3.8 cm (1.5 in) diameter rubber fire hose, was attached to a ball valve and enabled the contents of the sampler to be discharged to the onsite disposal system. Also a part of this module were a depth recording device with stilling wells and a high- and low-level float switch.

Module 2 was comprised of the electrical controls for the system. A single control box contained the timers, relays and pump controls which provided for sequencing the operation of the float switches, grinder pump, sample and discharge valves, and the distributor system (Module 3). Two continuous chart recorders were also part of this module. One was linked to the depth recording device (Cole-Parmer Millivolt Recorder Model 8357-02), while the

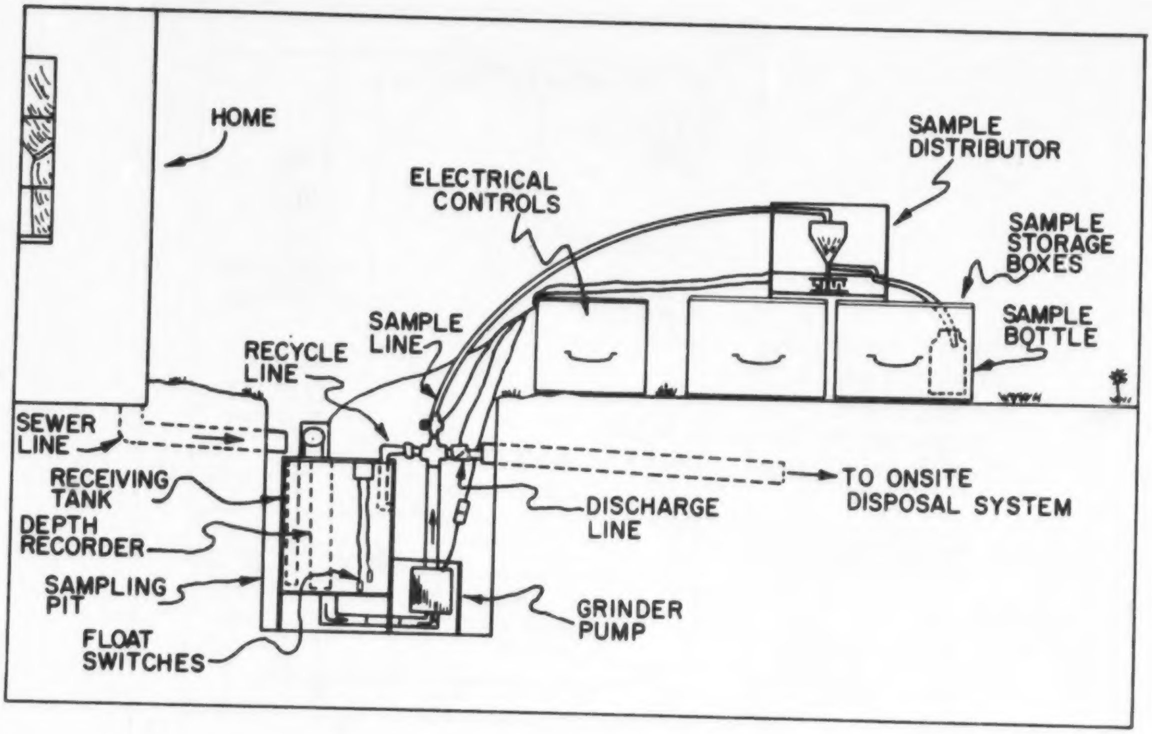
HOME
SAMPLE DISTRIBUTOR
ELECTRICAL CONTROLS
SAMPLE STORAGE BOXES
SAMPLE LINE
SAMPLE BOTTLE
RECYCLE LINE
SEWER LINE
RECEIVING TANK
TO ONSITE DISPOSAL SYSTEM
DISCHARGE LINE
DEPTH RECORDER
SAMPLING PIT
GRINDER PUMP
FLOAT SWITCHES

Figure A-4. Wastewater sampling system.

other was connected to a temperature probe (Cole-Parmer Temperature Minigraph Model 8356-41).

Module 3 included a unique sample distributor and sample storage boxes. A schematic of the distributor system is shown in Figure A-5 and a brief discussion follows.

The basic structure of the distributor included two 38 cm (15 in) square plexiglas plates held horizontally rigid several cm apart. A ball-bearing was pressed into a hole located in the center of each plate through which passed a 0.6 cm (0.25 in) vertical rod. To the upper end of this rod was fastened a 2 L funnel, which had a 1 cm (0.375 in) diameter arm of copper tubing projecting out perpendicular to the funnel axis.

The upper plexiglass plate also contained 24 sample ports spaced equally about a circle defined by the end of the arm attached to the funnel. The funnel and arm were rotated from sample port to port by a simple system. Two 23 cm (9 in) diameter plexiglas plates were located between the two square

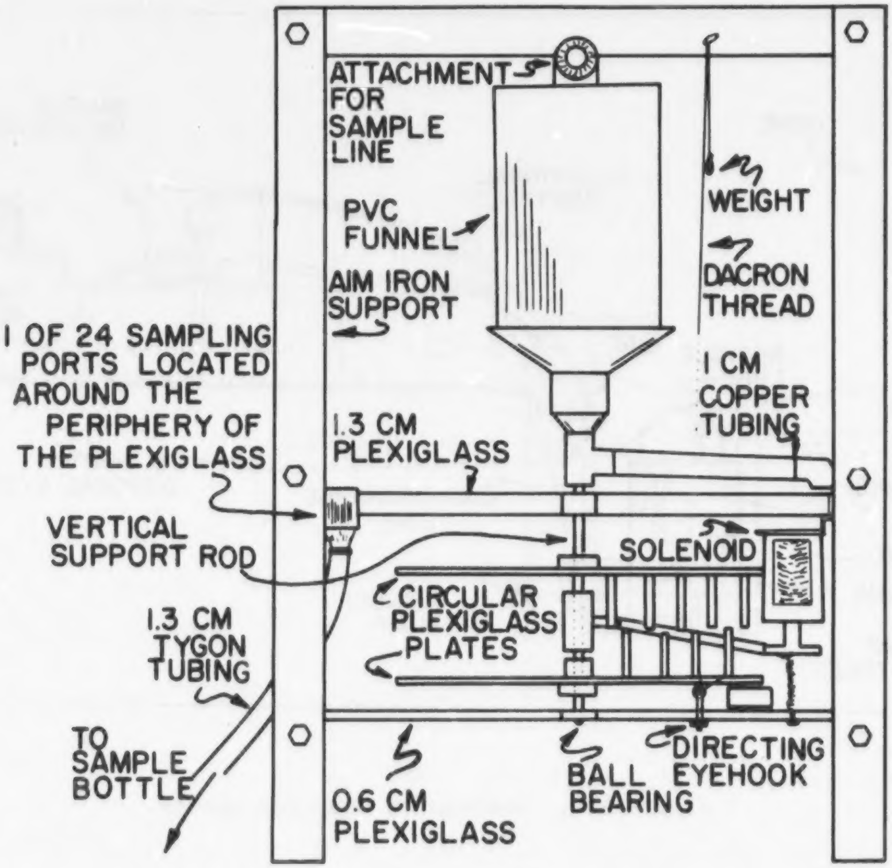
ATTACHMENT FOR SAMPLE LINE
PVC FUNNEL
AIM IRON SUPPORT
WEIGHT
DACRON THREAD
I OF 24 SAMPLING PORTS LOCATED AROUND THE PERIPHERY OF THE PLEXIGLASS
I CM COPPER TUBING
1.3 CM PLEXIGLASS
VERTICAL SUPPORT ROD
SOLENOID
CIRCULAR PLEXIGLASS PLATES
1.3 CM TYGON TUBING
TO SAMPLE BOTTLE
0.6 CM PLEXIGLASS
BALL BEARING
DIRECTING EYEHOOK

Figure A-5. Distributor system schematic.

plates and were rigidly fastened to the 0.6 cm vertical rod. Both of these circular plates contained 24, 2.5 cm (1.0 in) pegs equally spaced about its periphery. The plates as attached to the vertical rod were separated by about 5 cm (2 in) with the pegs facing toward each other and being exactly offset. Located between the two circular plates was a bushing through which the vertical rod passed freely, and to which was attached a brass rod (stepper arm) which could be moved up and down freely. This stepper arm passed between two adjacent pegs on the lower plate and was connected to a solenoid attached vertically to the upper, 38 cm (15 in) square plexiglas plate.

A heavy thread was attached to one of the pegs on the lower plate and directed to a weight in such a way that a rotating force would be exerted on the vertical support rod, the circular plexiglas plates and pegs, and the funnel and arm. When inoperative, the end of the funnel arm would be positioned directly over one of the 24 sample ports. Activating the solenoid would raise the stepper arm into the pegs on the upper plate and allow the funnel arm to rotate until a peg on the upper plate hit the stepper arm. Deactivating the solenoid caused the stepper arm to drop back down into the lower pegs, allowing the funnel arm to rotate until the next lower peg hit the stepper arm. At this point, the funnel arm was positioned above the next sample port. In this manner, the funnel arm was rotated to each of the 24 sample ports. Rotation past the 24th sample port closed a switch which disconnected the sample valve of module one and also prevented further rotation.

Insulated storage boxes were provided with this module to preserve the collected samples. The distributor system was positioned on top of these boxes and lengths of 1.3 cm (0.5 in) diameter Tygon tubing were attached to each of the 24 distributor sample ports and directed through holes in the boxes into one of 24, 2L sample bottles. Each of the sample ports corresponded to a certain sample bottle position so that with the ports numbered in order of rotation of the funnel arm, the sample bottles would be numbered in the order in which they were taken.

When installed at one of the study homes as shown in Figure A-4, the wastewater generated was directed into the polyethylene receiving tank. When the volume of wastewater in the tank reached 13 L (3.5 gal) or more, the high level float switch was activated which in turn initiated a sampling cycle. (Note: samples of water-using events producing less than 13 L were not obtained.) The grinder pump was activated first, homogenizing and recycling the contents of the receiving tank for approximately 90 sec. Then, the sample valve was opened to permit a 1 to 2 L sample to be pumped to the distributor system. Fifteen seconds later, the discharge valve was opened and the contents of the receiving tank were discharged to the onsite disposal system. When the low-level float switch was activated, the pump operation was terminated. The remainder of the cycle included opening the sample valve briefly to gravity drain the sample line back into the receiving tank (170 sec), the advancement of the distributor system to the next sample port (205 sec), closing of the discharge valve (280 sec) and resetting of the system (300 sec).

To determine whether a sample taken by this system was representative of the wastewater flowing into the receiving tank, several mixing tests were performed. The results of these tests indicated that the system did take representative samples of the influent wastewater.

Data collection--During operation at each home, daily site visits were made. The sample bottles were replaced, the distributor system reset, and the unit inspected. Any samples collected were transported back to the University of Wisconsin for analysis. The sampling system was moved between each of the four study homes until each had been monitored over several 3- to 4-day periods. In total, data were collected for 35 days.

Data interpretation and analysis--Each sample collected was analyzed for various chemical/physical parameters according to the procedures listed in Table A-15. The event that produced each collected sample was identified by several means. The depth recording device and continuous chart recorder enabled the determination of the time of occurrence and volume of the event which generated each of the collected samples. The residents of each home cooperated during each monitoring period by completing a check sheet noting the type of event and time of occurrence. Also helpful in identification were visual inspections of the samples. Finally, an added aid were the flow patterns generated by the depth recorder, which were found to be characteristic of each of the water-using events. Utilizing these means, the responsible event was identified for about 50 percent of the collected samples. Of these, approximately 80 percent were utilized. The remaining 20 percent of the samples were generated by more than one water-using event and were therefore not used in the analysis.

TABLE A-15. LABORATORY ANALYSIS PROCEDURES

Parameter	Test Employed
BOD_5 Unfiltered BOD_5 Filtered	Two Yellow Springs Instrument Company dissolved oxygen meters used on each sample. Meters were standardized using Winkler method with azide modification. For the filtered sample a Whatman #2 filter was used.
TOC Unfiltered TOC Filtered	Beckman Model 915 total organic carbon analyzer. For the filtered sample a Whatman #2 filter was used.
Total solids and Volatile solids	According to procedures outlined in Section 224A and 224B of Standard Methods (1971).
Suspended solids and Volatile Suspended Solids	According to procedures outlined in Section 148C of Standard Methods (1971). Measurements made directly using a 2.1 cm glass fiber filter disk.
Total Nitrogen	Olson Modified Semimicro Kjeldahl procedure (Bremner, 1965a).
NH_3-N and NO_3-N	Steam distillation-titration procedure (Bremner and Keeney, 1965).
Total Phosphorus and Ortho Phosphorus	Vanadomolybdate Yellow Color Method (Jackson, 1958).
Grease	After above samples taken off, remainder of sample acidified with HCL and grease determined according to Soxhlet Extraction Method as listed in Standard Methods (1971).

To compile the results of the sample analyses, a computer program was developed. A principal function of the program was to make corrections for a 2.5 L (0.7 gal) residual left in the sampler receiving tank after each sample was taken. Further, utilizing the results of analyses of tap water samples obtained from each home, the program subtracted the carriage water contributions of the measured parameters. The corrected concentrations (mg/L) for each sample were subsequently converted to mass/capita/day values by utilizing the measured volume for the sample producing event and its frequency of occurrence (number/capita/day). As part of the first phase of this study, the frequency of occurrence for the bath/showers, clotheswasher and toilet usage were identified. The toilet event frequencies were separated into fecal and nonfecal by using information presented by Perry (1954). An average frequency for garbage disposal usage, kitchen sink usage, and dishwashing were based on information presented by Ligman, Hutzler and Boyle (1974) along with consultation with the homeowners. As a final operation on the data, the computer program performed a statistical analysis, yielding the mean mg/cap/day, range and standard deviation for each of the events studied.

Wastewater Quality Characterization: Microbiological--

A series of ancillary wastewater quality studies were conducted to identify the microbiological characteristics of two household events, bathing and clothes washing. The automatic sampling system previously described was not used to obtain samples for this characterization due to the high degree of in-line contamination which would have been present. Instead, samples were taken from each event by the household residents according to prescribed procedures. Basically, a sample was taken after the occurrence of an event prior to discharge of the wastewater to the sewer line. The samples were refrigerated until they were picked up and transported back to the University of Wisconsin laboratories for analysis (within 24 hours).

The initial sampling effort was conducted at three of the previously described study homes (G, I, C´ listed in Table A-14). Samples were obtained from the bath/shower, clotheswasher-wash cycle, and clotheswasher-rinse cycle and analyses were performed for total and fecal coliforms and fecal streptococci. In addition, coliform and streptococcal isolates were taken for further characterization.

Analyses for fecal and total coliform organisms were conducted using the membrane filter technique of Standard Methods (1971) with the following variations: sterile 0.05% peptone in distilled water was used as diluent; after each filtration two, 20-30 mL portions of sterile diluent were used to rinse the sample dilution bottle and filter funnel; and counts were expressed as averages of duplicate samples or sample dilutions filtered. Fecal streptococci were enumerated by the pour plate technique of Standard Methods (1971) or by membrane filtration when sample volumes were 10 mLs or greater.

Coliform and streptococcal isolates were taken from plates of the above media and placed on slants of tryptone glucose yeast extract agar. Coliform isolates were taken, streaked on plates of Levine's Eosin Methylene blue agar to check colonial morphology, reactions and purity; necessary re-isolations and gram stains followed. Streptococcal isolates were handled similarly using streak plates of m-enterococcus agar. Hydrogen peroxide (3%) was applied to these plates to check for catalase. The media and tests used for further characterization of coliform and streptococcal isolates are given in Tables A-16 and A-17, respectively.

Further sampling was conducted at three additional households (Table A-18). Samples were obtained from bathing and clothes washing (wash cycle only) events utilizing similar sampling procedures, and analyses were performed for total and fecal coliforms, fecal streptococci, *Pseudomonas aeruginosa* and *Staphylococcus aureus*.

TABLE A-16. MEDIA AND TESTS - COLIFORM ISOLATE CHARACTERIZATION

Medium	Test
Lactose Broth	Acid and gas production at 35°C at 24, 48 and 72 hours.
EC Broth	Growth and gas in 24 hours at 44.5°C.
Tryptophane Broth	Indole production (I) in 24 hours at 35°C.
Buffered Glucose	Acid production in 5 days at 35°C, methyl red indicator (M) --- acetylmethyl carbinol production in 2 days at 35°C (Vi).
Simon's Citrate Agar	Use of citrate (C) as sole carbon source.
Combination tube of ornithine decarboxylase medium and H_2S-motility medium	H_2S production (H), ornithine decarboxylation (o) and motility (M_o); test read in 24 hours.

TABLE A-17. MEDIA AND TESTS USED - STREPTOCOCCAL ISOLATE CHARACTERIZATION

Medium	Test
Brain Heart Infusion Broth (BHI)	Growth at 10°C and 45°C.
6.5% NaCl Broth	Salt tolerance - growth in 48 hours.
0.1% Methylene Blue Milk	Tolerance and growth - methylene blue reaction.
Lactose Broth	Acid Production.
Starch Agar	Starch Hydrolysis.
Arginine Broth	Arginine hydrolysis with NH_4 production in 24 hours.
Bile Exculin Agar	4% bile tolerance and osculin hydrolysis.
BHI plus tellurite (1 part tellurite in 2560 parts BHI)	Tellurite tolerance.
Litmus Milk	Reactions over a 14-day period.

TABLE A-18. ADDITIONAL FAMILY INFORMATION

Family Unit	Adults	Children (Age)	Bathrooms	Automatic Clothes Washer	Automatic Dish Washer	Garbage Disposal	Occupation of Head of Household
L	2	2 (4 mo., 7)	1	Yes	Yes	No	Poultry Man
M	2	0	1	No	No	No	Poultry Man
N	2	0	1 1/2	Yes	Yes	Yes	Engineer

Analyses for fecal and total coliform organisms were conducted using the 5 tube MPN test procedures outlined in Standard Methods (1971) with the following variations: sterile .05% peptone in distilled water was used as a diluent. Fecal streptococci were enumerated by the pour plate technique of Standard Methods (1971). Analyses for Pseudomonas aeruginosa included growth at 37°C in 5 tube MPN of asparagine enrichment broth. A blue-green fluorescent pigment is produced by P. aeruginosa and inocula from the positive tubes were transferred to slants of acetamide agar and incubated for 24 hours at 37°C. A confirmation of positives was performed at this point by preparing streak plates on King's A agar and looking for blue-green pigment production after incubation at 37°C for 24 hours. Staphylococcus aureus counts were determined by a modification of Standard Methods (1971). The membrane filter technique was utilized with the filtered samples pre-enriched on a tryptone yeast broth. The medium used contained mannitol, 7.5% sodium chloride, and 0.75 M sodium azide as selective agents. After incubation for 72 hours at 37°C, suspicious colonies were screened for DNASE production and confirmed to be S. aureus by the coagulase test.

A final sampling effort was conducted at home N (Table A-18) of several wastewaters generated through hygienic care of an infant who had just received an oral polio vaccination. In-house samples were obtained by the adult residents from the baby's stools, diaper pail, bath and laundry over a twelve-day period. Where appropriate, each event was characterized as to quantity or volume, contributing clothing, additives, and so forth. Analyses were conducted for total and fecal coliforms, fecal streptococci, Pseudomonas aeruginosa, and virus infectivity.

Analyses for bacteriological characteristics were conducted as described previously. Virus infectivity was determined utilizing the following plaque assay procedure. Approximately 10 mL of the original sample was transferred to a test tube to which Diethyl ether (0.5 mL) was added. The test tube was held for 1 hour at 22°C to decontaminate the sample bacteriologically (for fecal samples, 24 hours at 4°C). Sterile air was then used to bubble off the ether. Then, 1 mL of fetal calf serum was added, the pH was adjusted to 9 using NaOH, and the prepared sample was sonicated in ice water for 5 minutes. Using 0.5 mL of the prepared sample, $\log_{10}$ serial dilutions were made. Two, freshly confluent 25 cm^2 flasks of Hela cell cultures were used per dilution, with 0.5 mL of each dilution inoculated into each of two flasks. The flasks were incubated at 22°C for two hours, with rocking every 15 minutes to wet the cells. After incubation, the inoculum was poured off, and the cell sheets in each flask were overlaid with 5 mL of the following:

Eagle's minimum essential media, 2 x	- 80%
Fetal calf serum (GIBCO)	- 8%
$NaHCO_3$ (5.6% solution)	- 4.8%
$MgCL_2$ (20% solution)	- 4%
Gentamicin (5 mg/mL solution; Schering)	- 1.6%
Fungizone (0.5 mg/mL solution; Squibb)	- 1.6%
Deionized water	- q.s.

3% Noble Agar, DIFCO

(Raise the media temperature to 42°C and lower the Agar temperature to 42°C. Mix the two components in the proportion of 1 part Agar to 3 parts media and maintain at 42°C until used.)

The flasks were held until the overlay media hardened on the cell sheet and then incubated with the cell side up for 60 hours at 37°C. After this incubation, the overlay is removed by adding about 10 mL of water to each flask and gently floating the overlay off the cell sheet by careful agitation. After pouring off the overlay, to each flask was added 3 mL of crystal violet stain of the following composition:

Ethanol (95% solution)	- 30 mL
Formalin (37% solution)	- 25 mL
Deionized water	- 345 mL
Crystal Violet	- 2.5 gram
NaCl	- 4.25 gram

The stain was left on the cells for 10 minutes and then rinsed off with water. The plaques present were then counted. The total number of plaques on four flasks of two adjacent dilutions (e.g. 10^{-4} and 10^{-5}) were divided by 1.1 and multiplied by the reciprocal of the highest $\log_{10}$ dilution to determine the plaque forming units per mL (PFU/mL).

Results and Discussion

Water Use Characterization--

Individual event water use--The water-use data collected were analyzed separately for each individual home and then summarized for all eleven homes combined. For each water use event, the average water use per occurrence and the number of occurrences per capita per day were calculated as shown in Tables A-19 and A-20. The daily water usage resulting from each of the events is presented in Table A-21. Inspection of these tables, readily reveals a significant variation in water use habits between the homes studied.

The daily per capita water use values determined in this study were compared with the results of previous investigators. As shown in Table A-22, with the exception of the toilet contribution, there is fair agreement between investigators regarding individual event water usage. The lower value for

TABLE A-19. WATER USED PER OCCURRENCE, L

Family Unit	Toilet		Laundry		Bath/Shower		Dishwasher		Water Softener	
	Mean	S.D.*	Mean	S.D.	Mean	S.D.	Mean	S.D.	Mean	S.D.
A	16.6	-	134	7.3	119	62.8	64.3	≈ 0	286	7.6
B	14.4	-	43	6.0	79	38.1	38.6	16.2	-	-
C	12.5	-	137	10.5	89	53.6	42.3	14.4	271	17.5
D	11.3	-	126	14.3	76	39.2	46.1	14.5	360	42.5
E	18.1	-	158	18.6	72	48.6	43.8	15.0	-	-
F	17.0	-	103	32.3	84	34.7	48.4	14.6	-	-
G	16.3	-	108	38.7	70	59.8	39.7	12.0	264	9.1
H	15.1	-	132	12.9	62	36.2	29.9	8.7	251	6.4
I	17.8	-	105	27.6	80	40.1	47.2	17.8	-	-
J	16.6	-	132	15.3	81	35.1	52.2	10.5	-	-
K	14.0	-	145	8.0	80	30.7	50.3	14.7	547	238
Weighted Average	15.1	-	127	26.7	81	44	47.2	14.7	307	121

* In all cases, the standard deviation for toilet flush was computed to be less than 1.5L.

TABLE A-20. FREQUENCY OF OCCURRENCE, uses/cap/day

Family Unit	Toilet	Laundry	Bath/Shower	Dishwasher	Water Softener
A	2.07	0.36	0.43	0.29	0.08
B	2.29	0.19	0.38	0.26	-
C	1.70	0.36	0.31	0.31	0.06
D	2.79	0.23	0.66	0.41	0.02
E	1.71	0.33	0.45	0.24	-
F	1.39	0.46	0.26	0.39	-
G	1.49	0.15	0.47	0.36	0.05
H	2.29	0.32	0.36	0.36	0.24
I	1.68	0.59	0.34	0.49	-
J	3.10	0.27	0.57	0.40	-
K	2.93	0.34	0.55	0.54	0.03
Weighted Average	2.29	0.31	0.47	0.39	0.03

toilet usage determined in this study is the result of a lower frequency of usage which is felt to have been caused by the residents using toilet facilities outside of their homes.

TABLE A-21. INDIVIDUAL EVENT WATER USAGE, L/cap/day

Family Unit	Toilet	Laundry	Bath/ Shower	Dish- Washer	Water Softener	Other	Total
A	34.4	47.6	51.0	18.9	23.0	39.3	214
B	32.9	8.3	30.2	10.2	-	15.1	96
C	21.2	49.5	27.6	13.2	15.1	20.4	147
D	31.8	28.7	50.3	18.9	8.7	17.0	155
E	31.0	52.9	32.8	10.6	-	29.9	157
F	23.4	48.0	21.5	18.9	-	15.5	127
G	24.6	15.9	32.9	14.0	12.9	12.5	113
H	34.8	42.3	22.3	11.0	59.3	18.5	188
I	29.9	61.2	26.8	23.0	-	16.6	158
J	52.2	35.9	46.1	20.8	-	15.9	170
K	41.2	49.1	44.2	26.8	17.8	35.9	215
Weighted Average	34.8	39.7	37.8	18.5	9.8	20.4	161
% of Daily Flow	21.6%	24.7%	23.5%	11.5%	6.1%	12.7%	100%

TABLE A-22. INVESTIGATOR COMPARISON OF EVENT WATER USAGE, L/cap/day

Event	Laak (1975)	Cohen and Wallman (1974)	Ligman, Hutzler and Boyle (1974)	Bennett and Linstedt (1975)	This Study
Toilet	74.8	65.0	75.6	55.6	34.8
Bath/Shower	32.1	23.8	47.2	32.9	37.8
Bathroom Sink	7.9		-	18.9	-
Kitchen Sink	13.6	68.0	-	9.8	-
Dishwashing	-		13.2	4.2	18.5
Garbage Grinding	-		5.7	3.0	-
Clothes Washing	28.0	39.7	37.8	43.8	39.7
Water Softening	-	-	-	-	9.8
Miscellaneous	-	-	-	-	20.4
Total	156	197	180	168	161

Total daily water use--The total per capita water use at each home is presented in Table A-23. As shown, the per capita usage was found to be quite variable at a given home, as evidenced by the large standard deviations and wide 90% confidence intervals determined. It is interesting to note, however, that the 90% confidence interval for all homes combined was relatively narrow, 154 to 168 L/cap/day (40.7 to 44.4 gal/cap/day) for a mean value of 161 L/cap/day (42.6 gal/cap/day).

TABLE A-23. DAILY PER CAPITA WATER USAGE, L/cap/day

Family Unit	Family Members	Days	Mean	S.D.	90% C.I.
A	4	28	214	98	185 - 245
B	3	14	96	47	76 - 117
C	4	77	147	88	130 - 163
D	6	42	155	47	143 - 167
E	3	28	157	113	122 - 192
F	5	28	127	71	105 - 150
G	7	35	113	45	100 - 125
H	3	24	188	55	169 - 206
I	5	28	158	61	139 - 177
J	7	68	170	43	162 - 179
K	4	62	215	112	192 - 239
Weighted Average	-	-	161	84	154 - 168

To illustrate the variation in per capita usage between homes, the mean and 90% confidence intervals for each are presented in Figure A-6.

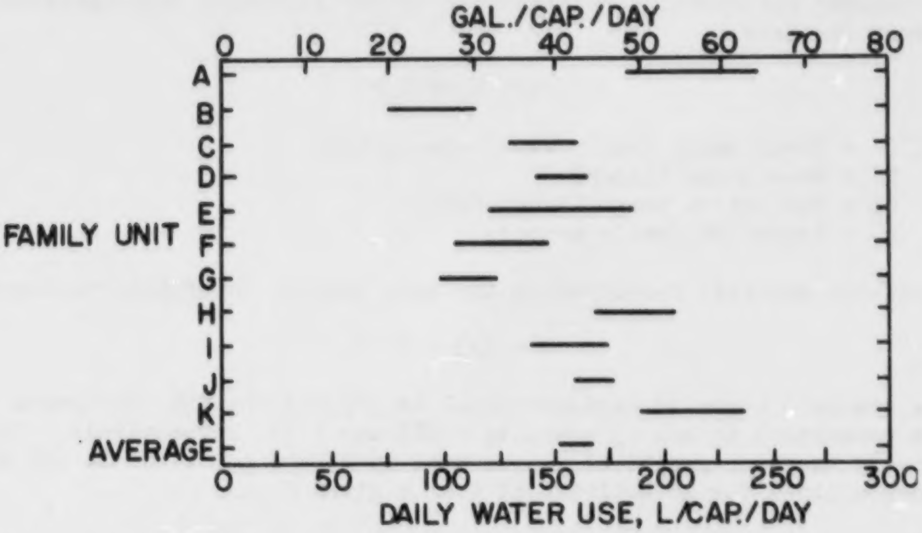

Figure A-6. Daily water usage.

As shown, the mean daily usage varied from a low of 96 L/cap/day (25.4 gal/cap/day) for Family B to a high of 215 L/cap/day (56.9 gal/cap/day) for Family K. Importantly, however, is the fact that for 8 of the 11 homes, the upper limit of the 90% confidence interval was less than 189 L/cap/day (50 gal/cap/day) and in no homes was it greater than 246 L/cap/day (65 gal/cap/day).

The daily per capita water use values determined in this study were compared to the results of earlier investigators. As shown in Table A-22, there is good agreement between investigators regarding average per capita water use, with the mean values varying from a low of 156 L/cap/day (41.3 gal/cap/day) to a high of 197 L/cap/cay (52.1 gal/cap/day).

Further to comparing the study averages, the mean daily water usage determined for each of the individual households involved in the studies by Cohen and Wallman (1974), Laak (1975) and Bennett and Linstedt (1975) were analyzed in combination with the results of this study. In total, the mean daily water use for each of 28 homes ranging in size from two to seven members was available for analysis. A statistical analysis of the per capita water use values yielded a mean of 180 L/cap/day (47.5 gal/cap/day) with a 95% confidence interval of 153 to 205 L/cap/day (40.6 to 54.3 gal/cap/day). Considering the variation in characteristics which existed for these 28 families, the 95% confidence interval is surprisingly narrow.

Although a variety of factors appear to influence the total quantity of water used within a home, family size has been suggested as a major factor. An analysis of the data from the 28 homes was conducted to determine if water usage could be more accurately predicted by a relationship including family size compared to that based on a per capita usage. The data were first plotted as total family water use versus family size (Figure A-7). Although the data were somewhat scattered, a relationship of the following form appeared to represent the data,

$$Q = K_1 + K_2P \qquad (1)$$

where, Q = Total daily family water use (L/day)
K_1 = Base usage (L/day)
K_2 = Per capita usage (L/cap/day)
P = Number of family members

A regression analysis conducted on the data yielded the following expression,

$$Q = 293 + 97\ P \qquad (2)$$

with a standard error of estimate equal to 189 and the 95% confidence limits on the parameters K_1 and K_2 equal to ± 225 and ± 52, respectively. This relationship has been graphically presented in Figure A-7 with the 90% and 95% confidence lines for an estimate of Q at a given P.

The two-variable relationship (2) appears to offer a more accurate estimate of average household water use for the smaller (2 member) and larger (5 to 7 member) family sizes than the simpler per capita expression (1). However, for family sizes of 3 to 4 members, the estimates provided by both are similar.

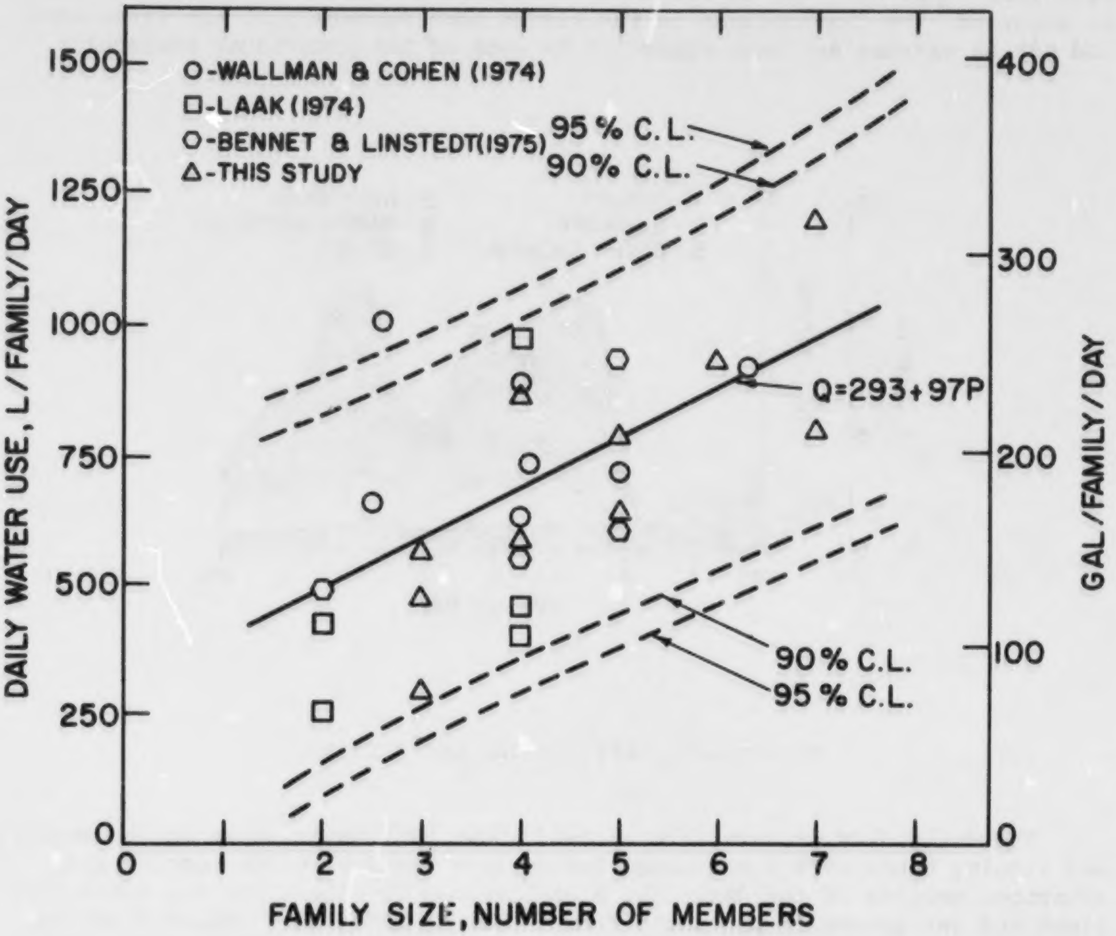

Figure A-7. Mean household water use versus family size.

Daily and weekly water use patterns--Utilizing the data generated in this study, graphs were constructed to illustrate household water use variations during the day and from day-to-day. The graphs presenting a summary of all eleven homes combined are presented in Figures A-8 and A-9. The graphs for each individual home may be found in Attachment A of this appendix. As would be expected, the fluctuations in the eleven home summary plot are attenuated and not as extreme as those exhibited by each of the individual households.

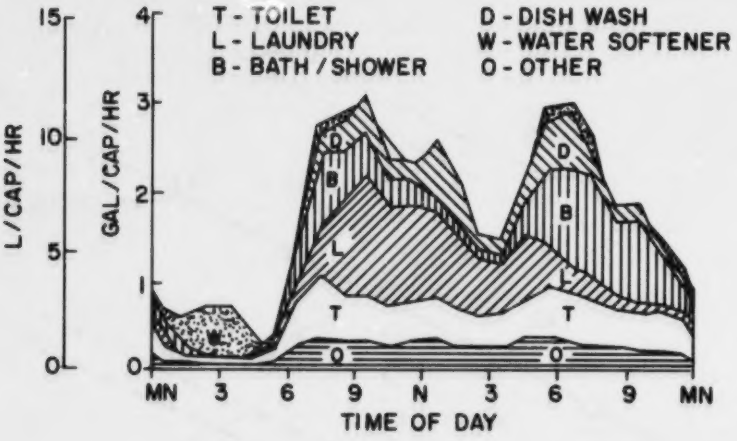

Figure A-8. Daily water use pattern.

The daily flow pattern (Figure A-8) shows high water usage in the morning and evening hours with lower usage during late evening, early morning and afternoon periods of the day. The miscellaneous or other flow was quite constant and was generally prevalent from 6 a.m. to midnight. Toilet flushing followed a similar pattern. Laundry was largely concentrated in the morning between 7 a.m. and 2 p.m., while baths and showers were most prevalent in the evening hours between 5 p.m. and midnight. Dishwashings were measured in three peaks following mealtimes, with the largest flow between 5 and 7 p.m. The water softener was concentrated between midnight and 5 a.m.

Little day-to-day variation in flow occurred for any of the events studied except bathing and clothes washing (Figure A-9). Bathing showed a significant difference between Friday (30.2 L/cap/day) and Saturday (45.7 L/cap/day). Clothes washing flows were significantly higher on Monday. When considering total daily flow, no one day was significantly different from the average.

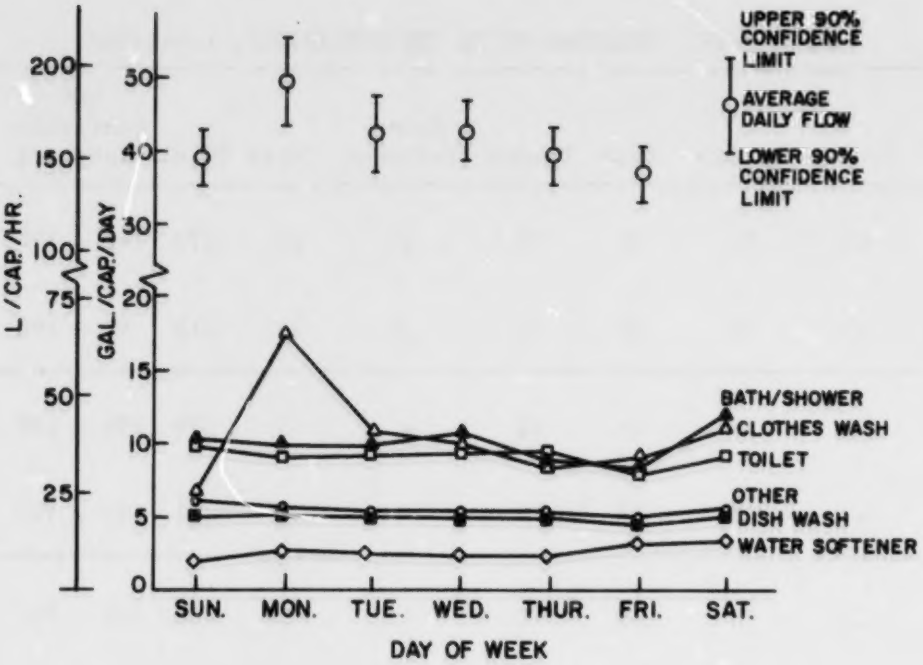

Figure A-9. Weekly water use patterns.

Seasonal variation--Water use data for three of the homes (C,J,K) were obtained during both the summer and winter seasons to allow an investigation of potential seasonal variations. It was believed that lifestyle changes between summer and winter, including daily school attendance by children during the winter, might result in a significant difference in water usage. A summary of the water usage for the homes as measured during the winter and summer seasons is presented in Table A-24. A review of the data collected in consultation with the homeowners led to the conclusion that households demonstrate widely varying characteristics and habits which are more important in determining water usage than the season of the year.

Wastewater Quality Characterization: Chemical/Physical--

Individual event wastewater-- Quality characteristics are reported as pollutant contributions by event occurrence and per capita per day with the mean results determined, presented in several ways: concentration per event, mass/event, mass/cap/day and percent contributed per event. In Table A-25, the average concentrations (mg/L) measured for each pollutant are presented. The mean mass (mg) of each pollutant produced by a single occurrence of each event are presented in Table A-26. In Table A-27 the daily per capita mass contributions (mg/cap/day) are shown, with the percentages of the total daily contribution presented in Table A-28.

TABLE A-24. SEASONAL WATER USE COMPARISON, L/cap/day

Family Unit	Toilet	Laundry	Bath	Dishes	Water Softener	Other	Total	90% Confidence Interval	Days
C Summer	23	53	32	15	15	24	170	146 - 194	42
C Winter	19	45	22	11	6	17	119	99 - 140	35
J Summer	57	35	45	21	-	17	170	159 - [illegible]	[illegible]
J Winter	44	38	48	21	-	19	171	158 - 183	26
K Summer	35	49	39	26	11	28	188	165 - 212	35
K Winter	49	49	51	28	26	46	250	207 - 293	27

TABLE A-25. MEAN POLLUTANT CONCENTRATIONS, mg/L

Parameter	Toilet Flush		Garbage Disposal	Kitchen Sink Usage	Automatic Dish Washer	Clotheswasher		Bath/ Shower
	Fecal	Nonfecal				Wash	Rinse	
BOD_5 U	610	330	1030	1460	1040	380	150	170
BOD_5 F	330	200	240	800	650	250	110	100
TOC U	500	220	690	880	600	280	100	100
TOC F	220	160	370	720	390	190	72	61
TS	1500	910	2430	2410	1500	1340	410	250
TVS	1090	610	2270	1710	870	520	180	190
TSS	880	320	1490	720	440	280	120	120
TVSS	720	260	1270	670	370	170	69	85
TOT-N	210	140	60	74	40	21	6	17
NH_3-N	84	27	.9	6	4.5	.7	.4	2
NO_3-N	.9	1.1	0	.3	.3	.6	.4	.4
TOT-P	38	14	12	74	68	57	21	2
Ortho-P	16	10	8	31	32	15	4	1
Temperature	66°F	66°	71°	80°	101°	90°	83°	85°
Flow*	16.3	16.3	14.4	18.1	45.4	59.3	54.4	49.1
Number of Samples	32-40	24-37	4-7	7-11	13-15	24-27	24-28	18-24

*Flow values were determined in the wastewater quality study and are in Liters.

TABLE A-26. MEAN POLLUTANT CONTRIBUTIONS, mg/event

Parameter	Toilet Flush		Garbage Disposal	Kitchen Sink Usage	Automatic Dish Washer	Clotheswasher		Bath/ Shower
	Fecal	Nonfecal				Wash	Rinse	
BOD_5 U	10000	5360	14600	26800	47500	22900	8210	8230
BOD_5 F	5390	3340	3430	14700	29500	14900	5820	4980
TOC U	8160	3520	9800	16000	27300	16400	5330	4680
TOC F	3640	2660	5240	13200	17600	11400	3920	3010
TS	24600	15000	34500	44200	68300	79800	22500	12200
TVS	17900	10100	32200	31200	39600	31200	9840	9560
TSS	14400	5280	21200	13200	19800	16900	6260	6020
TVSS	11700	4300	18100	12300	16700	10000	3750	4190
TOT-N	3460	2230	850	1350	1820	1250	330	840
NH_3-N	1380	440	13	110	210	42	22	99
NO_3-N	15	18	0	6	14	36	22	20
TOT-P	620	230	170	1350	3090	3400	1140	99
Ortho-P	260	160	110	560	1460	900	220	49

TABLE A-27. MEAN PER CAPITA POLLUTANT CONTRIBUTIONS, mg/cap/day

Parameter	Toilet Flush		Garbage Disposal	Kitchen Sink Usage	Automatic Dish Washer	Clotheswasher		Bath/ Shower
	Fecal	Nonfecal				Wash	Rinse	
BOD_5 U	4340	6380	10900	8340	12600	10800	4010	3090
BOD_5 F	2340	3980	2570	4580	7840	6970	2840	1870
TOC U	3530	4250	7320	5000	7280	7700	2610	1750
TOC F	1580	3170	3910	4110	4690	5380	1910	1130
TS	10700	17800	25800	13800	18200	37500	10900	4590
TVS	7760	12000	24000	9730	10500	14700	4800	3600
TSS	6240	6280	15800	4110	5270	7930	3040	2260
TVSS	5090	5120	13500	3840	4460	4700	1810	1580
TOT-N	1500	2640	630	420	490	580	150	310
NH_3-N	590	520	9.6	32.3	54	19.4	11.4	40
NO_3-N	6.3	21.1	.2	1.8	4.1	17	10.3	7.4
TOT-P	270	280	130	420	820	1600	550	36
Ortho-P	120	190	90	180	380	410	110	20

TABLE A-28. MEAN PER CAPITA POLLUTANT CONTRIBUTIONS, % OF TOTAL DAILY

Parameter	Toilet Flush Fecal	Toilet Flush Nonfecal	Garbage Disposal	Kitchen Sink Usage	Automatic Dish Washer	Clotheswasher Wash	Clotheswasher Rinse	Bath/ Shower
BOD_5 U	8.8	12.9	-	16.8	25.5	21.7	8.1	6.2
BOD_5 F	7.7	13.1	-	15.0	25.8	22.9	9.3	6.2
TOC U	11.0	13.2	-	15.6	22.7	24.	8.1	5.4
TOC F	7.2	14.4	-	18.7	21.4	24.5	8.7	5.1
TS	9.4	15.7	-	12.1	16.0	33.1	9.6	4.1
TVS	12.3	19.0	-	15.4	16.7	23.3	7.6	5.7
TSS	17.8	17.9	-	11.7	15.	22.5	8.7	6.4
TVSS	19.1	19.3	-	14.4	16.8	17.7	6.8	5.9
TOT-N	24.6	43.5	-	7.0	8.0	9.5	2.4	5.0
NH_3-N	46.7	41.0	-	2.5	4.2	1.5	.9	3.2
NO_3-N	9.3	31.0	-	2.6	6.0	25.	15.2	10.9
TOT-P	6.8	7.0	-	10.6	20.6	40.3	13.8	.9
Ortho-P	8.2	13.4	-	12.6	27.2	29.2	7.9	1.5

* Garbage disposal results are not included.

It should be emphasized that (1) all results presented were based on direct measurement except for mass/cap/day values which necessarily included some assumptions concerning event frequencies of occurrence, and (2) the contributions of pollutants from the carriage waters were removed from all results.

The results shown in the mean value tables (Tables A-25 to A-28) indicate how the mean concentrations and mass loadings contributed vary between the different types of household events. This variation in event wastewater quality is as expected, based on the variable nature and origin of the wastewaters.

A statistical analysis was conducted for each type of event and the mg/cap/day contributions of the various parameters. These results are included in Attachment B of this appendix. Based on this analysis, the dispersions about the mean values were found to be significant as evidenced by large standard deviations and wide ranges. For example, the mean mg/cap/day unfiltered BOD_5 loading from the bath/shower event based on 22 samples was 3090 with a standard deviation of 2140 and a range of 790 to 6940 mg/cap/day. This is as expected in light of the variation in day-to-day habits at a given home and the variation in life styles between homes.

The mean results of this study were reviewed on an individual event basis and compared to the results obtained by earlier investigators. The comparisons were rather scant in many cases, since many of the parameters measured in this study were not reported in the earlier studies.

Toilet flushing--The separation of the toilet flush event into a fecal flush and a nonfecal flush was possible through visual inspection of the toilet flush samples. The fecal flush contributed lower mass/capita/day loadings than the nonfecal flush, principally because the latter occurred approximately 2.6 times as often. The total output from the toilet (fecal and nonfecal flushes) was found to contribute 21.7% of the unfiltered BOD_5, 35.7% of the suspended solids, 68.1% of the total nitrogen and 13.8% of the total phosphorus produced daily by a given home (Table A-28). When compared to the results of earlier investigators, the mean mg/cap/day values determined in this study were in general, found to be substantially lower (Table A-29). However, when compared on a mg/event basis, as in Table A-30, the results were in fair agreement. The method of determining the toilet wastewater concentrations and daily per capita contributions is believed to be the cause of this. Most of the earlier studies (Laak, 1975; Ligman, Hutzler and Boyle, 1974; and Bennett and Linstedt, 1975) determined the pollutant contributions from the toilet from a small number of analyses of individual samples of urine and feces, medical literature characterizing human waste products and user estimates of event frequency. The single study which actually sampled raw toilet wastewaters (Olsson, Karlgren and Tullander, 1968), also employed user estimates of event frequency to determine mass/cap/day contributions. In contrast, the mass/cap/day contributions determined in this study were based on actual on-site sampling of toilet wastewaters as well as measurement of event frequencies.

TABLE A-29. TOILET FLUSH WASTEWATER - INVESTIGATOR COMPARISON, mg/cap/day

Parameter	Olsson, Karlgren and Tullander (1968)	Laak (1975)	Ligman, Hutzler and Boyle (1974)	Bennett and Linstedt (1975)	This Study
BOD_5	20000	23500	23600	6900	10700
TSS	30000	-	30900	36500	12500
TOT N	11000	14500	16800	5200	4140
TOT P	1600	2110	1360	-	550

TABLE A-30. TOILET FLUSH WASTEWATER - INVESTIGATOR COMPARISON, mg/event

Parameter	Olsson, Karlgren and Tullander (1968)	Laak (1975)	Ligman, Hutzler and Boyle (1974)	Bennett and Linstedt (1975)	This Study
BOD_5	4000	4360	6380	1920	6740
TSS	6000	-	8340	10100	7870
TOT N	2200	2680	4540	1470	2600
TOT P	320	390	370	-	340
uses/cap/day	5.0	5.4	3.7	3.6	1.6

Dishwashing--The wastewater produced from the kitchen sink was the result of manual dishwashing and major dish rinsing (pots and pans) at homes having automatic dishwashers. Thus, the total mass/cap/day contributions from dishwashing are represented by the sum of the kitchen sink and automatic dishwasher contributions. Dishwashing proved to be a major contributor of pollutants, generating 42.3% of the unfiltered BOD_5, 26.7% of the suspended solids and 15% of the total nitrogen and 31.2% of the total phosphorus. When compared to the values reported by earlier investigators, the results of this study were found to be significantly higher (Table A-31). However, most of the reported values of earlier investigators are within the range of values determined in this study. The discrepancy in the BOD_5 results, as well as in the results for the other parameters, were most likely caused by the normal differences in the life style and dishwashing habits of the families whose homes were sampled.

TABLE A-31. DISHWASHING WASTEWATER - INVESTIGATOR COMPARISON, mg/cap/day

Pollutant	Olsson, Karlgren and Tullander (1968)	Laak (1975)	Ligman, Hutzler and Boyle (1974)	Bennett and Linstedt (1975)	This Study
BOD_5	17000	9200	5900	11100	21000
TSS	13000	-	2700	2200	9380
TOT N	600	50-400	-	1100	910
TOT P	300	-	450	-	1240
Contributing Events:	Kitchen Area	Manual Dish-washing	Manual Dishwashing	Dishwasher, Kitchen Sink	Dish-washer Kitchen Sink

Garbage disposal--The garbage disposal results presented in this study are based on the analysis of samples taken from the garbage disposal wastewaters produced by homes without automatic dishwashers. The results obtained in this study were lower than expected, based on earlier analyses performed by Ligman, Hutzler and Boyle (1974) and Bennett and Linstedt (1975) (Table A-32). An explanation as to the reason for this may be found in the fact that the families which had garbage disposals in this study, also had large dogs. Consultation with the homeowners revealed that a majority of the meal scraps which might otherwise have been disposed of through the garbage disposal were fed to the dogs. Since the use of garbage disposals in rural homes served by

TABLE A-32. GARBAGE DISPOSAL WASTEWATER - INVESTIGATOR COMPARISON, mg/cap/day

Parameter	Ligman, Hutzler and Boyle (1974)	Bennett and Linstedt (1975)	This Study
BOD_5	30900	12300	10900
TSS	43600	20200	15800
TOT N	910	200	630
TOT P	-	-	130
uses/cap/day	0.75	0.40	0.75

individual sewage disposal systems is discouraged and since the results obtained for the garbage disposal in this study were based on a limited number of samples, the garbage disposal results were omitted when calculating the total mass/cap/day loadings from a typical rural household and the percentages contributed by the individual events.

Clothes washing--Based on the results obtained in this study, the household operation of washing clothes proved to be a major contributor of pollutants. On a mass/cap/day basis, the automatic clothes washer contributed 29.8% of the unfiltered BOD_5, 31.2% of the suspended solids, 11.9% of the total nitrogen and 54.1% of the total phosphorus (Table A-28). In each case, approximately 70% of the pollutants were contained in the wash-cycle discharge with the remaining 30% in the rinse-cycle discharge (Table A-26). The results obtained in this study (wash and rinse cycles combined) were compared to those of earlier investigators and were found to be somewhat higher on a mg/cap/day basis (Table A-33). When compared on a mg/event basis, however, the results are in fair agreement. The discrepancy in per capita contributions is due to the different event frequencies used in their computation.

TABLE A-33. LAUNDRY WASTEWATER - INVESTIGATOR COMPARISON, mg/cap/day

Parameter	Olsson, Karlgren and Tullander (1968)	Laak (1975)	Ligman, Hutzler and Boyle (1974)	Bennett and Linstedt (1975)	This Study
BOD_5	3000	7900	9500	8700	14800
TSS	2000	-	7260	3400	11000
TOT N	200	-	-	200	730
TOT P	1300	-	2270	-	2150
uses/cap/day	?	0.30	0.25	0.30	0.48

Bath/Shower--The results for the bath/shower event, based on samples of individual bath and shower events grouped together, proved it to be a minor contributor of pollutants. On a daily basis, this event contributed the lowest percentage of almost all pollutants measured: 6.2% of the unfiltered BOD_5, 6.4% of the suspended solids, 5.0% of the total nitrogen and less than 1.0% of the total phosphorus (Table A-28). The pollutant contributions determined in this study were found to be within the range of values reported by earlier investigators (Table A-34).

Daily pollutant contributions--Of prime importance to many individuals are the total daily pollutant contributions of an individual within a typical household. The results determined in this study, as well as those reported by previous investigators, for BOD_5, suspended solids, phosphorus and nitrogen are presented in Table A-35. It should be noted that the pollutant contributions of household garbage disposals have been excluded from the results presented. It is interesting to note how closely the results generated by the various investigators agree. As noted previously, the lower nitrogen contribution determined in this study is for the most part due to the frequency of toilet usage.

TABLE A-34. BATH/SHOWER WASTEWATER - INVESTIGATOR COMPARISON, mg/cap/day

Parameter	Laak (1975)	Ligman, Hutzler and Boyle (1974)	Bennett and Linstedt (1975)	This Study
BOD_5	6180	9100	3200	3090
TSS	-	5450	900	2260
TOT N	-	-	0	310
TOT P	-	-	-	40
uses/cap/day	?	0.51	0.32	0.39

TABLE A-35. DAILY POLLUTANT CONTRIBUTIONS,* gram/cap/day

Pollutant	Olsson, Karlgren and Tullander (1968)	Ligman, Hutzler and Boyle (1974)	Laak (1975)	Bennett and Linstedt (1975)	This Study
BOD_5	45.0	48.1	48.6	34.8	49.6
Suspended Solids	48.0	46.3	-	47.3	35.1
Nitrogen	12.1	16.8	-	7.2	6.1
Phosphorus	3.8	4.1	-	-	4.0

* Garbage disposal contributions have been excluded.

The use of a garbage disposal can substantially increase the quantities of pollutants shown in Table A-35. Based on investigations by Ligman, Hutzler and Boyle (1974), Bennett and Linstedt (1975), and the results of this study, the increase in pollutant contributions that can be expected due to the use of a garbage disposal have been outlined in Table A-36. As shown, the use of a garbage disposal dramatically increases the contribution of BOD_5 and suspended solids while adding little additional nitrogen and phosphorus.

TABLE A-36. INCREASE IN POLLUTANT MASS DUE TO GARBAGE DISPOSALS, grams/cap/day

Pollutant	Ligman, Hutzler and Boyle (1974)	Bennett and Linstedt (1975)	This Study
BOD_5	30.9 (64%)*	12.3 (35%)	10.9 (22%)
Suspended Solids	43.6 (94%)	20.2 (43%)	15.8 (45%)
Nitrogen	.9 (5%)	.2 (3%)	.6 (10%)
Phosphorus	-	.1 (3%)	.1 (3%)

* Percentage increase of the corresponding value shown in Table A-35.

Hourly pollutant distributions--Since individual water-using events occur intermittently and contribute varying quantities of pollutants, the strength of the wastewater generated from a home fluctuates during the day. To illustrate patterns for the fluctuations of various pollutants, the mass/cap/day results of this phase of the study were combined with the results of the water use characterization phase. In determining the hourly distribution of various pollutants, it was assumed that the mass/cap/day generated by an event was, on the average, distributed evenly in the daily flow from the event. The percentage of the daily flow generated during a given hour from a given event was multiplied times the mean mg/cap/day loading of a given pollutant to determine the mass of the pollutant produced during the hour in question. This was done for the toilet, automatic clothes washer, bath/shower and dishwashing events for each hour of a typical day for BOD_5, suspended solids, total nitrogen and total phosphorus, as shown in Figures A-10 through A-13. It should be noted that these graphs are similar to the summary water use patterns developed (Figures A-8, A-9) and the fluctuations, in this case in wastewater quality, for an individual home are likely to be considerably greater than those shown in the summary graphs.

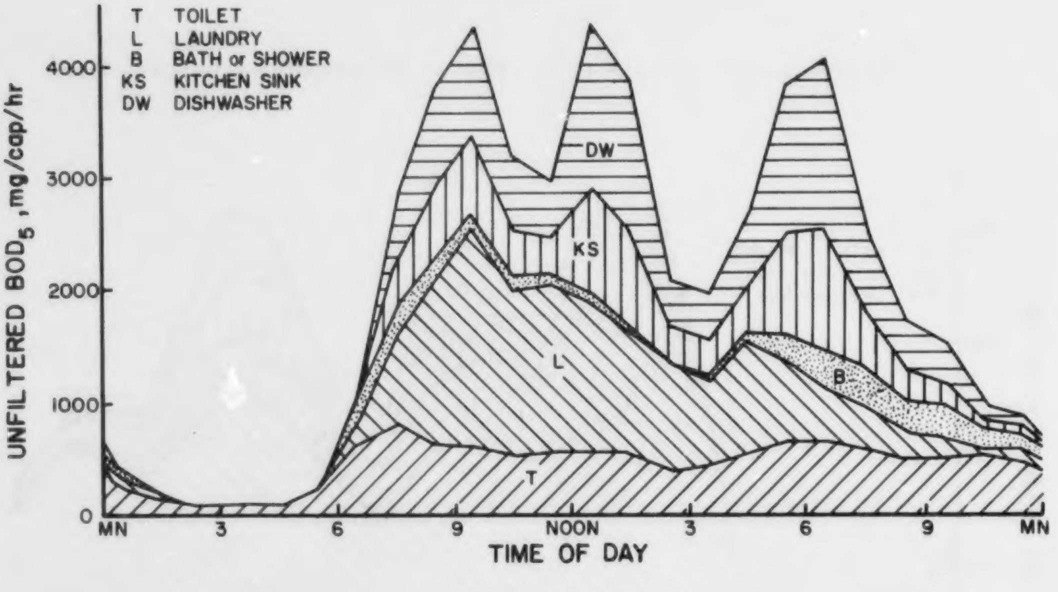

Figure A-10. Hourly distribution of BOD_5.

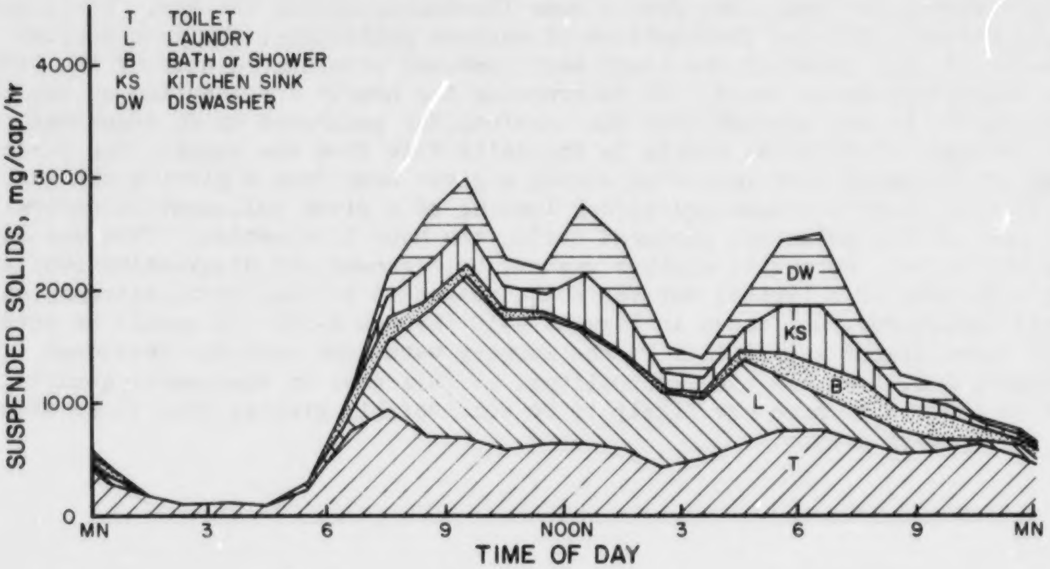

Figure A-11. Hourly distribution of suspended solids.

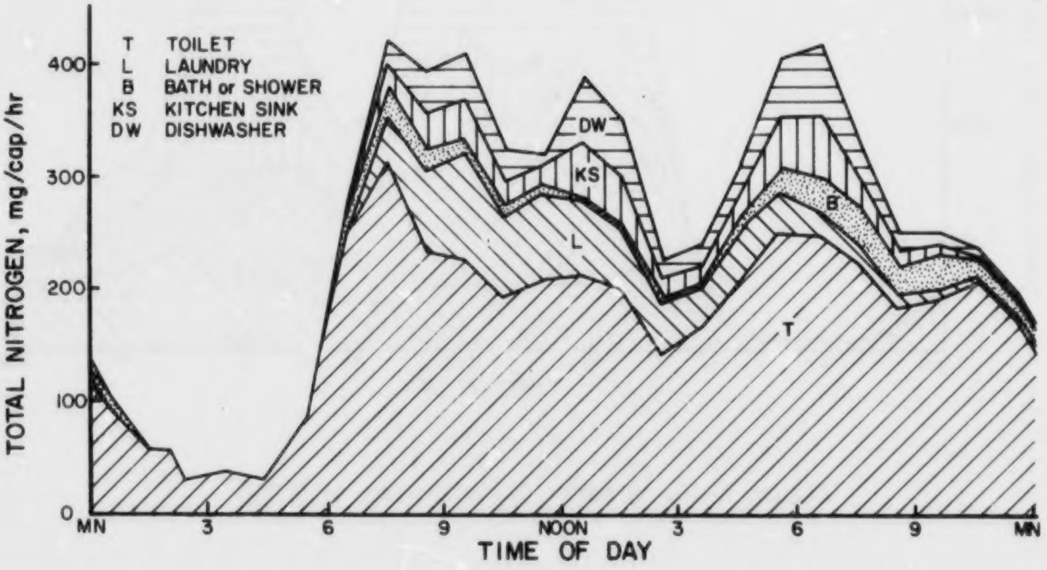

Figure A-12. Hourly distribution of total nitrogen.

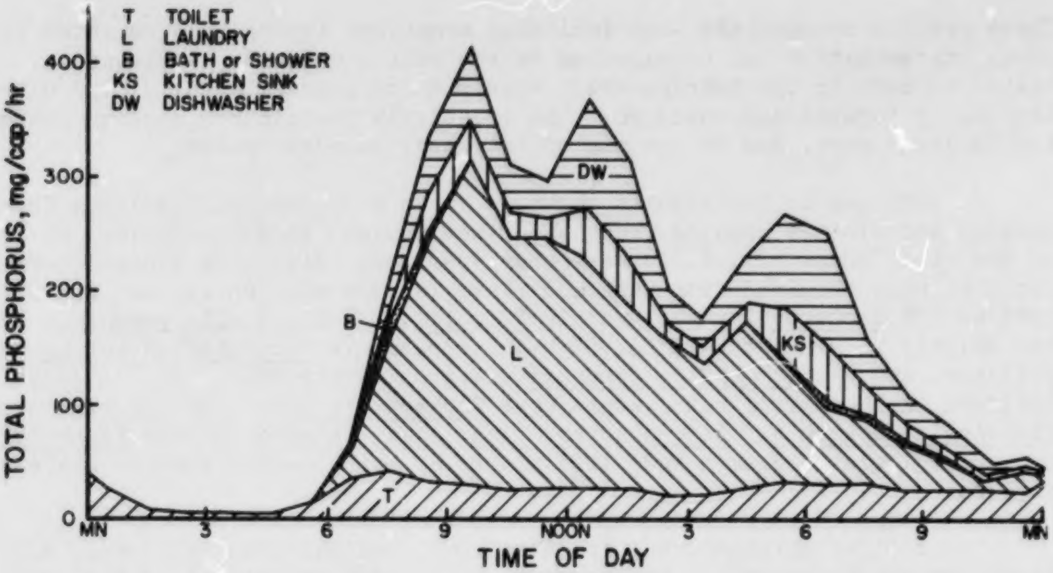

Figure A-13. Hourly distribution of total phosphorus.

Wastewater Quality Characterization: Microbiological--

The results of analyses for total and fecal coliforms and fecal streptococci on bathing and clothes washing samples obtained from six study households are summarized in Table A-37.

TABLE A-37. BACTERIOLOGICAL CHARACTERISTICS OF BATH AND LAUNDRY WASTEWATERS

Event	Organism	Samples	Mean* #/100 mL	Confidence Intervals,* #/100 mL 95%	99%
Clothes† Washing	Total Coliforms	41	215	65 - 700	45 - 1020
	Fecal Coliforms	41	107	39 - 295	28 - 405
	Fecal Streptococci	41	77	27 - 220	19 - 305
Bathing	Total Coliforms	32	1810	710 - 4600	530 - 6160
	Fecal Coliforms	32	1210	450 - 3240	330 - 4410
	Fecal Streptococci	32	326	100 - 1050	70 - 1510

* Log-normalized data.

† Samples were obtained from the middle of the wash cycle. Samples taken from 15 rinse cycles were consistently lower than the corresponding wash cycle values.

These results demonstrate that indicator organisms typically associated with fecal contamination can be expected in raw bath and laundry wastewaters. The higher numbers in the bathing event appear to be primarily associated with the bathing of infants and children. The relatively low clothes washing numbers are in large part, due to the use of hot water laundry cycles.

In addition to the results shown in Table A-37, several isolates from the bathing and clothes washing (wash and rinse cycles) samples obtained at three of the study homes (C', G, I) were characterized. Sixty-one fecal coliform isolates were obtained from wash and rinse laundry wastewaters and characterized as 65% *Escherichia* spp. (mainly *E. coli*), 27% *Klebsiella pneumonia* (with the ability to grow at 44.5°C), 5% high temperature *Enterobacter aerogenes* biotypes, and 2% *Citrobacter freundii*. Approximately 90% of the 24 fecal coliform isolates from bath waters were *Escherichia* spp. with the remainder, *Klebsiella pneumonia*. *Enterobacter*, *Klebsiella*, *Citrobacter* and *Escherichia* spp. were isolated from m-Endo (TC) plates of bath, wash and rinse wastewater samples.

Forth-eight streptococcal isolates were obtained from bath, wash and rinse wastewater samples. Enterococci made up 38% of these isolates; the majority of the bath enterococci were *S. faecalis* var. liquefaciens, whereas only a few of the enterococcal isolates taken from clothes wash and rinse wastewaters were of this species. Twenty-two percent of streptococcal isolates were characterized as *S. bovis*. Other streptococcal species generally found on and in the body of animals and man (Viridens and Pyogenic groups) were also isolated.

Much of the bacterial contamination in these bath and clothes washing wastewaters was probably from the natural environment or natural skin flora of man as indicated by the incidence of *S. faecalis* var. *liquifaciens*, *S. bovis* and other nonfecal streptococcal isolates found. Many of these organisms, though associated with animal feces, are often considered to exist in nature and probably have less sanitary significance than other enterococcal species. However, the high incidence of *E. coli*, *Klebsiella* and enterococci, especially in wash and rinse wastewaters, indicates that these wastewaters possess a "potential" for fecal contamination.

Analyses were performed on the samples obtained from the remaining three study homes (L, M, N) for two common potential pathogens, *Pseudomonas aeruginosa* and *Staphylococcus aureus*. The results as shown in Table A-38 indicate a very low incidence of *Pseudomonas aeruginosa* and in those samples where it was isolated, the concentrations were always less than 20/100 mL. *Staphylococcus aureus* was not isolated in any of the samples analyzed.

The results of the in-house sampling at Home N of wastewaters generated through hygienic care of an infant who had just received an oral polio vaccination are summarized in Table A-39. In addition to the infant sample results shown several samples were obtained from the bathing and clothes washing events of the two adult residents. These latter samples yielded total and fecal coliform and fecal streptococci concentrations within the range of values determined previously (Table A-37).

TABLE A-38. PSEUDOMONAS AERUGINOSA AND STAPHYLOCOCCUS AUREUS IN BATH AND LAUNDRY WASTEWATER

		Bathing			Laundry		
Organism	Home	Samples	Positive Samples	Highest Value	Samples	Positive Samples	Highest Value
Pseudomonas aeruginosa	L	10	2	2/100 mL	17	5	20/100 mL
	M	1	0	*[a]	4	0	*[b]
	N	10	0	*[b]	5	0	*[b]
Staphylococcus aureus	L	9	0	*[c]	17	0	*[c]
	M	1	0	*[d]	4	0	*[e]
	N	10	0	*[c]	4	0	*[e]

* Below detention limit of test which was: [a]2/100 mL; [b]20/100 mL; [c]10/100 mL for 3 samples and 10^4/100 mL for the remaining; [d]10^4/100 mL; [e]10/100 mL.

To properly interpret the data shown in Table A-39 requires consideration of the relationship of the events sampled. This relationship was established based on in situ measurements and is graphically depicted in Figure A-14. The diapers containing stool samples were thoroughly rinsed in the toilet prior to their disposition in a diaper pail containing approximately 7 liters of water and one tablespoon of baking soda. Thus, the majority of the fecal material on the diapers was flushed down the toilet. Prior to this rinsing, a sample of the fecal material was taken. Diapers soiled by urine alone were deposited directly into the diaper pail. After 24-48 hours of collecting diapers in the pail, the diapers were swished around in the pail and a portion of the liquid fraction was poured off into the toilet (about 3 liters from which a sample was taken). The remaining liquid and diapers were deposited into the clothes washer (17 to 35 diapers and approximately 4 liters of liquid). Two of the laundry samples included a few baby clothing items also. The cleaning product used was 3/4 cup of Dreft. Prior to the end of the laundry wash cycle, a sample of the washer contents was taken. Samples of the infant's bath water were taken after bathing prior to discharge of the bath water.

As shown in Figure A-14, the laundry effectively yielded a one log reduction in the concentration of organisms measured in the diaper pail by dilution alone. However, as shown in Table A-39, total and fecal coliforms were reduced by about 8 logs, fecal streptococci were reduced by about 2 logs, and virus were reduced by about 2 logs. These reductions were probably due in large part to the "hot" laundry cycles which were routinely used (temperature = 60°C). Of special note, are the very low levels of indicator bacteria and the absence of virus in all of the laundry-wash cycle samples. The analysis of bathing samples yielded concentrations within the range of values determined previously for the selected indicator bacteria. Surprisingly, virus was only isolated in one of five samples at a low level of 5 PFU/mL.

TABLE A-39. MICROBIOLOGICAL CHARACTERISTICS OF INFANT RELATED WASTEWATERS*

Event	Total Coliforms Log no./ 100 mL	Fecal Coliforms Log no./ 100 mL	Fecal Strep. Log no./ 100 mL	Pseudomonas aeruginosa Log no./ 100 mL	Virus Infectivity PFU/mL
Diaper Pail	8.71	8.67	2.65	<1.30	2.55
	(6)	(6)	(6)	(6)	(7)
	7.43-9.98	7.40-9.94	1.58-3.73	-	2.17-2.93
Bathing	3.93	3.93	4.48	<1.30	-
					1 sample -
	(4)	(4)	(4)	(4)	0.70
					4 samples -
	1.95-5.92	1.95-5.92	3.25-5.71	-	none detected
Laundry	0.38	0.38	<1.0	<1.30	6 samples -
(wash cycle)	(5)	(5)	(5)	(4)	none detected
	0.15-0.60	0.15-0.60	-	-	
Stool	11.01	10.94	11.64	<2.08	5.86
Sample†	(7)	(7)	(7)	(6)	(8)
	10.17-11.86	10.16-11.72	11.39-11.88	-	5.36-6.35

* Log normalized data; mean
(number of samples)
95% CI

† Stool sample values expressed per wet gram.

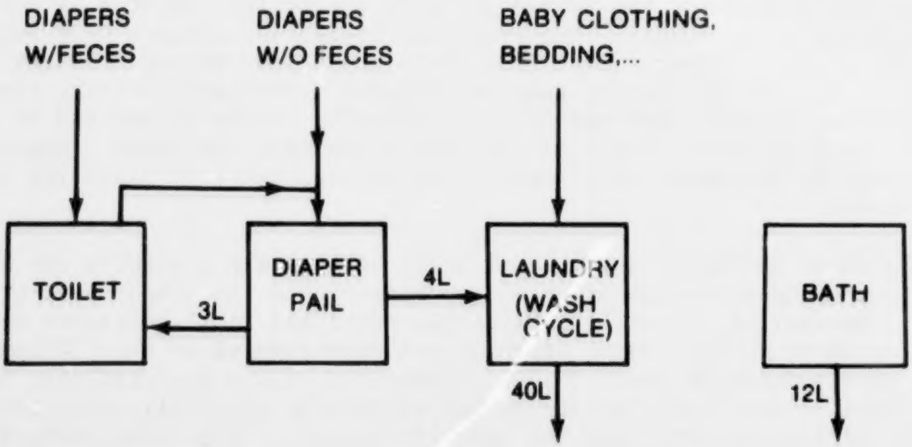
DIAPERS W/FECES
DIAPERS W/O FECES
BABY CLOTHING, BEDDING,...
TOILET
DIAPER PAIL
LAUNDRY (WASH CYCLE)
BATH
3L
4L
40L
12L

Figure A-14. Infant related wastewater sample relationships.

CHARACTERISTICS OF WASTEWATERS GENERATED BY RURAL ESTABLISHMENTS AND PUBLIC FACILITIES

The rural population, as well as the transient population moving through the rural areas, require the services offered by a variety of commercial establishments and facilities. As a result, these establishments and facilities are commonly located in unsewered areas and are forced to rely on some form of private sewerage for disposal of their wastewaters. Obviously, the list of establishments/facilities which could potentially be located in the rural areas is a lengthy one. However, certain establishments appear to be located in rural areas only infrequently, while others appear sufficiently similar to a residential household in terms of their waste producing sources that their wastewater characteristics should likewise be similar. Of special interest are those establishments/facilities which appear to occur frequently in unsewered locations and generate wastewaters whose characteristics may be considerably different from that of a typical household. The establishments selected for this investigation include:

Bars/Taverns
Bowling alleys
Campgrounds and picnic parks
Churches
Country (golf) clubs
Laundromats
Marinas
Motels
Restaurants
Schools
Service stations
Shopping centers
Sports facilities
Theaters

Initially, a major effort was expended to compile a comprehensive summary of existing water use/wastewater production characterization data. Subsequently, methodologies for estimating wastewater production at facilities serving transient populations were considered and evaluated as to their feasibility and appropriateness, and the existing characterization data base was expanded where necessary and feasible.

Summary of Existing Information

To determine the extent of the present data base for each of the establishments under investigation, a review of the literature was conducted and inquiries were made to various possible sources of information. A listing of the literature searched and a summary of those individuals and organizations contacted either by telephone or letter may be found in Attachment C of this Appendix. Based on the literature reviewed and the contacts made, a comprehensive summary of the existing information was compiled.

General Guideline Information--

A substantial number of references were found to contain guidelines for estimating the characteristics of wastewater produced by various public establishments. Lengthy tables such as the one found in the Manual of Septic Tank Practice (1967) were found in many standard texts, equipment manufacturers' catalogs and government regulatory codes. These guideline tables usually only present suggested daily flows, but a few also list BOD_5 and suspended solids contributions and the duration of flow. A few representative tables are shown in Tables A-40 through A-43.

TABLE A-40. QUANTITIES OF SEWAGE FLOWS
(Manual of Septic Tank Practice, 1967)

Type of Establishment	Gal. Per Person Per Day* (Unless Otherwise Noted)
Airports (per passenger)	5
Apartments - multiple family (per resident)	60
Bathhouses and swimming pools	10
Camps:	
Campground with central comfort stations	35
With flush toilets, no showers	25
Construction camps (semi-permanent)	50
Day camps (no meals served)	15
Resort camps (night and day) with limited plumbing	50
Luxury camps	100
Cottages and small dwellings with seasonal occupancy	50
Country clubs (per resident member)	100
Country clubs (per non-resident member present)	25
Dwellings:	
Boarding houses	50
additional for non-resident boarders	10
Luxury residences and estates	150
Multiple family dwellings (apartments)	60
Rooming houses	40
Single family dwellings	75
Factories (gallons per person, per shift, exclusive of industrial wastes)	35
Hospitals (per bed space)	250+
Hotels with private baths (2 persons per room)	60
Hotels without private baths	50
Institutions other than hospitals (per bed space)	125
Laundries, self-service (gallons per wash, i.e., per customer)	50
Mobile home parks (per space)	250
Motels with bath, toilet, and kitchen wastes (per bed space)	50
Motels (per bed space)	40
Picnic parks (toilet wastes only) (per picknicker)	5
Picnic parks with bathhouses, showers and flush toilets	10
Restaurants (toilet and kitchen wastes per patron)	10
Restaurants (kitchen wastes per meal served)	3
Restaurants additional for bars and cocktail lounges	2
Schools:	
Boarding	100
Day, without gyms, cafeterias, or showers	15
Day, with gyms, cafeteria, and showers	25
Day, with cafeteria, but without gyms, or showers	20
Service stations (per vehicle served	10
Swimming pools and bathhouses	10

(continued)

TABLE A-40 (continued)

Type of Establishment	Gal. Per Person Per Day* (Unless Otherwise Noted)
Theaters:	
Movie (per auditorium seat)	5
Drive-in (per car space)	5
Travel trailer parks without individual water and sewer hook-ups (per space)	50
Travel trailer parks with individual water and sewer hook-ups (per space)	100
Workers:	
Construction (at semi-permanent camps)	50
Day, at schools and offices (per shift)	15

* L/person/day = 3.8 x gal/person/day.

TABLE A-41. ESTIMATED WATER CONSUMPTION*
(Metcalf and Eddy, Inc., 1972)

Type of Establishment	Gal. Per Day Per Person or Unit†
Dwelling units, residential‡:	
Private dwellings on individual wells or metered supply	50-75
Apartment houses on individual wells	75-100
Private dwellings on public water supply, unmetered	100-200
Apartment houses on public water supply, unmetered	100-200
Subdivision dwelling on individual well, or metered supply, per bedroom	150
Subdivision dwelling on public water supply, unmetered, per bedroom	200
Dwelling units, treatment:	
Hotels	50-100
Boarding houses	50
Motels, without kitchens, per unit	100-150
Lodging houses and tourist homes	40
Camps:	
Pioneer type	25
Children's, central toilet and bath	40-50
Day, no meals	15
Luxury, private bath	75-100
Labor	35-50
Trailer with private toilet and bath, per unit (2-1/2 persons)*	125-150
Restaurants (including toilet):	
Average	7-10
Kitchen wastes only	2-1/2-3

(continued)

TABLE A-41 (continued)

Type of Establishment	Gal. Per Day Per Person or Unit[†]
Short order	4
Short order, paper service	1-2
Bars and cocktail lounges	2
Average type, per seat	35
Average type, 24-hr, per seat	50
Tavern,per seat	20
Service area, per counter seat (toll road)	350
Service area, per table seat (toll road)	150
Institutions:	
Average type	75-125
Hospitals	150-250
Schools:	
Day, with cafeteria or lunch room	10-15
Day, with cafeteria and showers	15-20
Boarding	75
Theaters:	
Indoor, per seat, two showings per day	3
Outdoor, including food stand, per car (3-1/3 persons)	3-5
Automobile service stations:	
Per vehicle served	10
Per set of pumps	500
Stores:	
First 25-ft frontage	450
Each additional 25-ft frontage	400
Country clubs:	
Resident type	100
Transient type, serving meals	17-25
Offices	10-15
Factories, sanitary wastes, per shift	15-35
Self-service laundry, per machine	250-500
Bowling alleys, per alley	200
Swimming pools and beaches, toilet and shower	10-15
Picnic parks, with flush toilets	5-10
Fairgrounds (based on daily attendance)	1
Assembly halls, per seat	2
Airport, per passenger	2-1/2

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[*] Water under pressure, flush toilets and wash basins are assumed provided unless otherwise indicated. These figures are offered as a guide; they should not be used blindly. Add for any continuous flows and industrial usages. Figures are flows per capita per day, unless otherwise stated.

[†] L/day = 3.8 x gal/day.

[‡] Add 125 gal. per trailer space for lawn sprinkling, car washing, leakage, etc.

TABLE A-42. WASTEWATER CHARACTERISTICS FOR PACKAGE TREATMENT PLANT SIZING (Goldstein and Moberg, 1973)

Type of Facility	Flow* (gal/ cap/ day	#5 Day B.O.D.† (lbs/cap/ day)	Runoff hours	Shock Load Factor
Airports - (per passenger)	5	.020	16	low
Airports - (per employee)	15	.050	16	low
Apartments - Multiple family	75	.175	16	med.
Boarding Houses	50	.140	16	med.
Bowling Alleys - per lane (no food)	75	.150	8	med.
Campgrounds - per tent or travel trailer site - central bathhouse	50	.130	16	med.
Camps - Construction - (semi-permanent)	50	.140	16	med.
Camps - Day (no meals served)	15	.031	16	med.
Camps - Luxury	100	.208	16	med.
Camps - Resort (night and day) with limited plumbing	50	.140	16	med.
Churches - per seat	5	.020	4	high
Clubs - Country (per resident member)	100	.208	16	med.
Clubs - Country (per nonresident member present)	25	.052	16	med.
Courts - Tourist or Mobile home parks with individual bath units	50	.140	16	med.
Dwellings - Single-family	75	.170	16	med.
Dwellings - Small, and cottages with seasonal occupancy	50	.140	16	med.
Factories - (gallons, per person, per shift, exclusive of industrial wastes. No showers.)	25	.073	8	high
Add for showers	10	.010		
Hospitals	250+	.518	16	med.
Hotels - with private baths (2 persons per room)	60	.125	16	med.
Institutions - other than hospitals (nursing homes)	125	.260	16	med.
Laundromat	400	varies	12	high
Motels - (per bed space)	40	.083	16	med.
Motels - with bath, toilet, and kitchen wastes	50	.140	16	med.
Offices - no food	15	.050	8	high
Parks - Picnic (toilet wastes only) (gallons per picnicker)	5	.010	8	high
Parks - Picnic, with bathhouses, showers, and flush toilets	10	.021	8	high
Restaurants - (kitchen wastes per meal served)	7	.015	8-12	high
Restaurants - (toilet and kitchen wastes per patron)	10	.021	8-12	high

(continued)

TABLE A-42 (continued)

Type of Facility	Flow* (gal/ cap/ day	#5 Day B.O.D.† (lbs/cap/ day)	Runoff hours	Shock Load Factor
Restaurants - additional for bars and cocktail lounges	3	.006	8-12	high
Schools - Boarding	100	.208	16	med.
Schools - Day, without cafeterias, gyms, or showers	15	.031	8	high
Schools - Day, with cafeterias, but no gyms or showers	20	.042	8	high
Schools - Day, with cafeterias, gyms, and showers	25	.052	8	high
Service Stations - (per vehicle served)	12	.021	16	med.
Shopping Centers - (no food - per sq. foot)	0.1		16	med.
Shopping Centers - (per employee)	15	.050	16	med.
Stores - (per toilet room)	400	.832	16	med.
Swimming pools and bathhouses	10	.021	8	high
Sports Stadiums	5	.020	4-8	very high
Theaters - Drive-in (per car space)	5	.010	6	high
Theaters - Movie (per auditorium seat)	5	.010	6	high
Trailer Parks - per trailer	150	.350	16	med.

* L/cap/day = 3.8 x gal/cap/day.

† g/cap/day = 454 x lbs/cap/day.

TABLE A-43. SUGGESTED DAILY FLOWS AND BOD CONSIDERATIONS (Goldstein and Moberg, 1973)

Class	Persons Per Unit	gal/cap/ day	lbs BOD_5/ cap/day Avg.	lbs BOD_5/ cap/day With Garbage Grinder	BOD_5 (mg/L)
Subdivisions, Better	3.5	100	0.17	0.25	205
Subdivisions, Average	3.5	90	0.17	0.23	220
Subdivisions, Low Cost	3.5	70	0.17	0.20	290
Motels, Hotels, Trlr. Pks.	2.5	50	0.17	0.20	400
Apartment Houses	2.5	75	0.17	0.25	225
Resorts, Camps, Cottages	2.5	50	0.17	0.20	400
Hospitals	per bed	200	0.30	0.35	200
Factories or offices	per person	20	0.06	-	360

(continued)

TABLE A-43 (continued)

Class	Persons Per Unit	gal/cap/ day	lbs BOD_5/ cap/day Avg.	lbs BOD_5/ cap/day With Garbage Grinder	BOD_5 (mg/L)
Factories, incl. showers	per person	25	0.07	-	340
Restaurants	per meal	5	0.02	0.06	450
Schools, Elementary	per student	15	0.04	0.05	320
Schools, High	per student	20	0.05	0.06	360
Schools, Boarding	per student	100	0.17	0.20	205
Swimming Pools	per swimmer	10	0.03	-	360
Theaters, Drive-in	per stall	5	0.02	-	450
Theaters, Indoor	per seat	5	0.01	-	250
Airports, Employees	per employee	15	0.05	-	450
Airports, Passengers	per passenger	5	0.02	-	480
Bars, Employees	per employee	15	0.05	-	450
Bars, Customers	per customer	2	0.01	-	800
Dairy Plants	per 1000# milk	100-250	0.56 to 1.66		650-2000
Public Picnic Parks	per picnicker	5-10	0.01	-	250
Country Clubs, Residents	per resident	100	0.17	0.25	205
Country Clubs, Members	per member	50	0.17	0.20	400
Public Institutions (non-hospital)	per resident	100	0.17	0.23	205

Most of the Water Pollution Control Agencies at the State level have established recommended and/or required guideline values for wastewater loadings from various commercial establishments and public service facilities. Typically, these state guidelines are limited to daily wastewater flow volumes but a few also include daily BOD_5 and suspended solids contributions and the flow period. Most states will allow deviations from their adopted guidelines, if the deviations can be proven accurate. The state guideline values for wastewater flow volume have been summarized in Table A-44.

The State guidelines appear to have been derived from a variety of sources, including standard textbooks, equipment manufacturers' literature and the Manual of Septic Tank Practice (1967). Sometimes information presented in these sources was reproduced exactly, but more often modifications were made based upon agency experience. The basis for the design guidelines used by a representative sample of the states is shown in Table A-45.

TABLE A-44. SUMMARY OF STATE FLOW GUIDELINES, gal/unit/day*

Establishment	Unit	No. of States	Mean	Mode	Range
Bars/Taverns	Seat	12	28	20(8)†	20-60
	Patron Space	6	5	2(3)	2-9
Bowling Alleys	Lane	17	96	75(8)	50-200
Picnic Parks (Toilet)	Capita	34	6	5(30)	5-10
Picnic Parks (Toilet, Showers)	Capita	29	12	10(24)	10-25
Campground (Central Bath House)	Capita	28	38	35(18)	25-50
	Site	5	100	-	50-150
Churches (No Kitchen)	Seat	16	4	5(7)	.5-5
(With Kitchens)	Seat	9	6	7(6)	.8-7.5
Country Clubs	Non-Resident Member	23	23	25(20)	15-37.5
	Resident Member	23	98	100(21)	75-100
	Member	13	48	50(9)	25-100
Laundromats	Machine	19	419	400(10)	150-800
	Wash	15	49	50(12)	40-50
Marinas	-	-	-	-	-
Motels	Bed Space	13	44	40(8)	40-65
	Room	18	109	100(12)	75-200
	Person	11	55	50(5)	40-100
Restaurants, Drive-in	Car Space	9	48	50(6)	5-100
Restaurants	Meal or Patron	22	10	10(8)	3-25
	Seat	19	38	35(7)	10-70
Schools (restrooms only)	Capita	37	14	15(23)	8-30
(restrooms plus cafeteria)	Capita	39	18	20(18)	8-30
(restrooms, cafeteria and showers)	Capita	39	23	25(16)	8-35
Service Station	Car	26	10	10(23)	5-15
	1st Bay	8	1000	1000(8)	1000-1000
	Each Added Bay	8	625	500(6)	500-1000
Shopping Centers	1000 ft^2	12	233	100(7)	100-1000
Stadia	Seat	7	3	3(5)	2-5
Theater, Drive-in	Car Space	40	6	5(29)	3-20
Theater	Seat	30	4	5(23)	2-5

* Liters = 3.8 x gal.

† Number in parenthesis equals the number of states using the value indicated for the mode.

TABLE A-45. SOURCE OF DESIGN CHARACTERISTICS USED BY VARIOUS STATES

State	Source of Design Characteristics
Alabama	Some literature, but mostly meter readings and experience.
Alaska	Manual of Septic Tank Practice (USPHS, 1967)
Arizona	Standard texts and catalogs.
Connecticut	Metcalf and Eddy or other standard texts.
Delaware	Developed own figures from other sources, based on experience. Have been found very reliable.
Georgia	Chrysler Corporation Equipment Catalog (based upon literature).
Idaho	Manual of Individual Water Supply Systems (USEPA, 1973)
Kentucky	Jet Aeration Company information.
Louisiana	Equipment manufacturers' catalogs.
Maine	Manual of Septic Tank Practice
Massachusetts	Flows used by neighboring states, modified when dictated by meter readings.
Michigan	Manual of Septic Tank Practice with modifications.
Minnesota	Manual of Septic Tank Practice
Missouri	Source unknown. Data used is satisfactory.
Nebraska	Davco Manufacturing Company. Data satisfactory.
New Hampshire	Federal guidelines and experience.
New Jersey	Standard textbooks.
New York	Source unknown.
North Dakota	Manual of Septic Tank Practice
Ohio	Jet Aeration Company.
Pennsylvania	Manual of Septic Tank Practice with modifications.
Rhode Island	Manual of Septic Tank Practice with modifications.
South Carolina	Literature review and actual sampling.
Tennessee	Previous literature, meter reading and experience.
Virginia	Mixture of studies and experiences.

Peak Flow Estimation--

Around 1940, Hunter developed a procedure for estimating water supply demands on plumbing systems (Hunter, 1940; 1941). Although originally developed for peak water demands and sizing distribution systems, it has been used to predict peak wastewater flows as well (WPCF MOP-9, 1970; IAPMO UPC, 1976). A discussion of this procedure, commonly referred to as the "Fixture-unit Method," follows.

The fixture-unit method, as developed, was based on the premise that in general types of application, a given type of fixture had an average flow rate and an average frequency and duration of use, which determined the water demand for the fixture. The fixture-unit was arbitrarily set equal to a flow rate of 7.5 gal/min and various fixtures were assigned a certain number of fixture units based upon their particular characteristics. Based on probability studies Hunter developed relationships between peak water demand and the number of fixture-units present. The fixture-units assigned to various fixtures by Hunter are listed in Table A-46 with curves relating peak demand to the total number of fixture-units present shown in Figure A-15.

TABLE A-46. FIXTURE-UNIT VALUES FOR VARIOUS FIXTURES (Hunter, 1941)

Fixture or Group	Occupancy	Type of Supply Control	Weight in Fixture Units
Water closet	Public	Flush valve	10
Water closet	Public	Flush tank	5
Pedestal urinal	Public	Flush valve	10
Stall or wall urinal	Public	Flush valve	5
Stall or wall urinal	Public	Flush tank	3
Lavatory	Public	Faucet	2
Bathtub	Public	Faucet	4
Shower head	Public	Mixing valve	4
Service sink	Office, etc.	Faucet	3
Kitchen sink	Hotel or restaurant	Faucet	4
Water closet	Private	Flush valve	6
Water closet	Private	Flush tank	3
Lavatory	Private	Faucet	1
Bathtub	Private	Faucet	2
Shower head	Private	Mixing valve	2
Bathroom group	Private	Flush valve for closet	8
Bathroom group	Private	Flush tank for closet	6
Separate shower	Private	Mixing valve	2
Kitchen sink	Private	Faucet	2
Laundry trays (1 to 3)	Private	Faucet	3
Combination fixture	Private	Faucet	3

Since its original development, the fixture-unit approach has been accepted and applied on a fairly wide-scale, not only for estimating peak water supply demands but also for predicting peak sewage flows. Descriptions of the approach including tables listing fixture-unit values and curves presenting peak flow versus total fixture units present, have been included throughout the literature (e.g. Manual of Septic Tank Practice, 1967; WPCF MOP-9, 1970; Salvatto, 1972; IAPMO UPC, 1976). Although variations exist between the values presented by various sources, the values presented in Table A-47 and the relationships shown in Figure A-16 appear to be representative.

The fixture-unit approach to estimating peak sewage flows possesses several potential shortcomings. First, it is based on average fixture characteristics and probability projections relating peak flows to total fixture units present. Deviations from the average values used in developing the approach could result in potentially significant variations from the peak flows predicted (WPCF MOP-9, 1970). For water use, Konen (1976) reports that in general, the estimated demand is typically 40% greater than that actually experienced. Further, in utilizing this approach, originally developed for water supply, to estimate peak wastewater flows, certain questionable assumptions must be made: (1) most water used exits the building as wastewater,

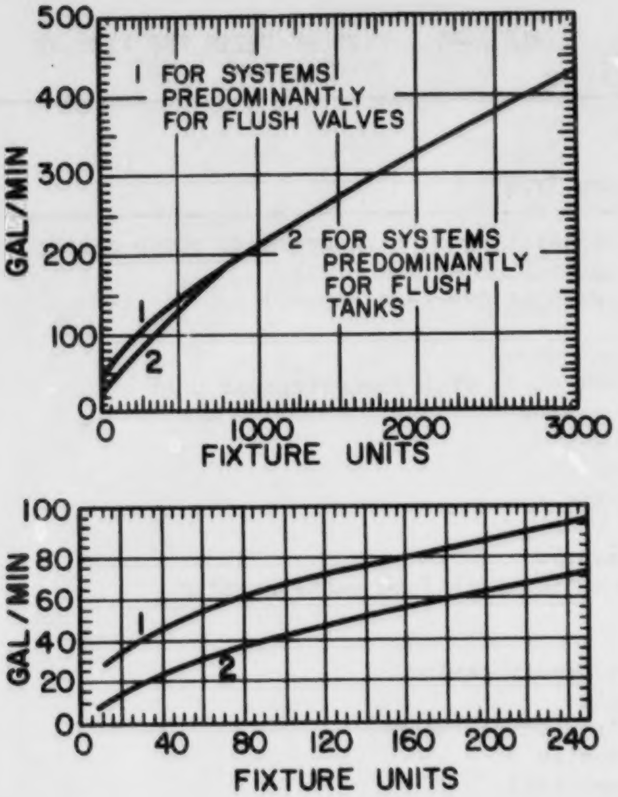


Figure A-15. Water supply demand versus fixture units present.

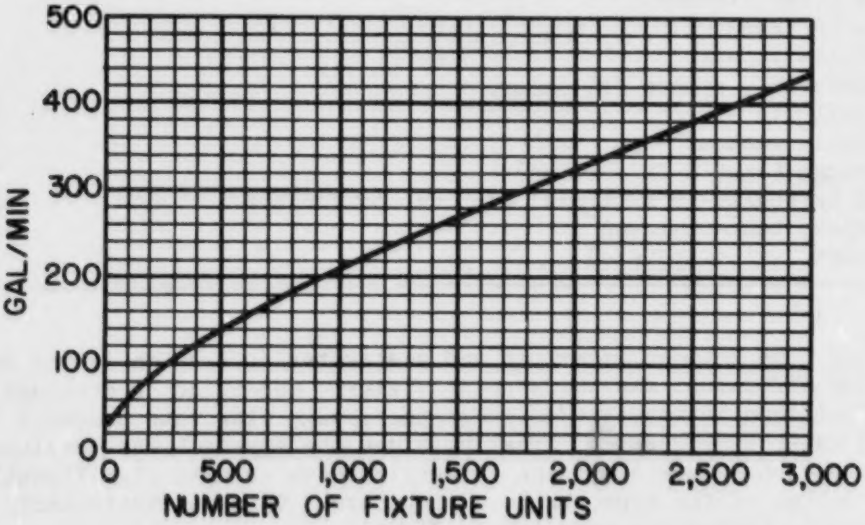


Figure A-16. Peak discharge versus fixture units present (WPCF MOP-9, 1970).

TABLE A-47. FIXTURE-UNITS PER FIXTURE
(WPCF MOP-9, 1970)

Fixture Type	Fixture-Unit Value as Load Factors
1 bathroom group consisting of tank-operated water closet, lavatory, and bathtub or shower stall	6
Bathtub (with or without overhead shower)	2
Bidet	3
Combination sink-and-tray	3
Combination sink-and-tray with food-disposal unit	4
Dental unit or cuspidor	1
Dental lavatory	1
Drinking fountain	1/2
Dishwasher, domestic	2
Floor drains	1
Kitchen sink, domestic	2
Kitchen sink, domestic, with food waste grinder	3
Lavatory	1
Lavatory	2
Lavatory, barber, beauty parlor	2
Lavatory, surgeon's	2
Laundry tray (1 or 2 compartments)	2
Shower stall, domestic	2
Showers (group) per head	3
Sinks	
Surgeon's	3
Flushing rim (with valve)	8
Service (Trap standard)	3
Service (P trap)	2
Pot, scullery, etc.	4
Urinal, pedestal, syphon jet, blowout	8
Urinal, wall lip	4
Urinal stall, washout	4
Urinal trough (each 2-ft section)	2
Wash sink (circular or multiple) each set of faucets	2
Water closet, tank-operated	4
Water closet, valve-operated	8

(2) the lag time between water use and wastewater production is not great, and (3) the effects of the water-using fixtures and building drainage systems do not significantly increase/decrease the sewage flow rate compared to the predicted water demand rate. Finally, since the approach was developed in 1940, fixture characteristics and lifestyles have changed significantly and a re-evaluation of the approach is appropriate. This is particularly true with regard to estimating peak sewage flows.

Individual Establishment Field Monitoring--

Field studies have been conducted on many of the establishments under study and have generated valuable information on wastewater characteristics. Unfortunately, the results of many of these studies have not received widespread distribution. In the following discussion, the results of actual field measurements on each of the establishments under study are presented.

Bars/Taverns--To date, it appears that no specific field studies have been conducted to determine the characteristics of bar or tavern wastewater. Several of the general guideline tables do, however, list daily flows and a few even suggest BOD_5 and suspended solids contributions (Tables A-40 through A-44). On a per patron basis, a typical flow is 7.6 L/day (2 gal/day) with a BOD_5 contribution of 4.5 g/day (0.01 lbs/day), while on a per seat basis the typical flow becomes 76 L/day (20 gal/day).

Bowling alleys--A limited amount of bowling alley water use information was generated by two studies around 1960. The Oakland County Department of Public Works in Michigan obtained meter readings from the local water department concerning each of 4 modest bowling alleys and determined the water use data shown in Table A-48 (Shah, 1976).

TABLE A-48. BOWLING ALLEY WATER USE (Shah, 1976)

Bowling Alley	Description	Dates	L/day/lane
A	17 Lanes	1954-1959	200
B	20 Lanes, Bar, 66 Seats	1957-1959	212
C	18 Lanes	1958	204
D	12 Lanes	1958	185

The Florida State Board of Health obtained water use information through monthly metering of a 24 lane bowling alley with restaurant and bar facilities (Santarone, 1976). The following water use data was generated:

January, 1960 - 21,830 L/day/lane
February, 1960 - 20,490 L/day/lane
March, 1960 - 33,990 L/day/lane
April, 1960 - 36,930 L/day/lane

Average Day = 950 L/day/lane
Maximum Day = 1230 L/day/lane
Minimum Day = 730 L/day/lane

The average flow of 950 L/day/lane is considerably higher than the 200 L/day/lane average determined by Oakland County. The higher flow/lane is difficult to explain without further information, but may be due to the restaurant facilities provided.

Guideline values for daily flows and BOD_5 contributions for bowling alleys (excluding food service) are typically in the range of 280 to 470 L/day/lane (75 to 125 gal/day/lane) and 59 to 145 g/day/lane (0.13 to 0.32 lbs/day/lane), respectively.

Campgrounds and picnic parks--As might be suspected, the United States Department of Agriculture-Forest Service has information regarding the characteristics of wastewaters produced through picnic and camping related activities (Kolzow, 1975). Based upon experience and characterization studies which they have conducted, the Forest Service has suggested system design loadings for average BOD_5 and average flow as shown in Table A-49. The results of analyses performed by the Forest Service on vault toilet wastes are summarized in Table A-50.

TABLE A-49. SUGGESTED DESIGN LOADINGS - CAMPING FACILITIES APPLICATION*(Kolzow, 1975)

Type of Use	Flow L/day/unit	BOD_5 g/day/unit
Camper		
No Showers Provided	130	45
Showers Provided	150	55
Picnicker	19	14
Swimmer		
No Showers Provided	19	14
Showers Provided	38	18
Picnic and Swim Area		
Participant	38	18
Boat Launch Area Participant	19	14
(Also Fisherman Parking)		

* Assumes toilets provided are conventional flush type.

TABLE A-50. VAULT TOILET WASTE ANALYSES (Kolzow, 1975)

Parameter	Average Concentrations (mg/L)
BOD_5	19,700
COD	40,300
Total Solids	44,700
Dissolved Solids	19,600
Suspended Solids	25,100

General guidelines for campgrounds and picnic parks may be grouped according to day-use parks with toilets only, day-use parks with toilets and showers, and campgrounds with a central bathhouse. For the day-use park providing only toilet facilities, on a per user basis, the typical flow is 19 L/day (5 gal/day) with a BOD_5 contribution of 4.5 to 13.5 g/day (.01 to 0.03 lbs/day). If shower facilities are also provided, the typical flow is 38 L/

day (10 gal/day) with a similar BOD_5. For a developed campground with a central bathhouse, an average flow of 150 L/camper/day (40 gal/camper/day) with a BOD_5 contribution of approximately 45.4 g/camper/day (0.1 lbs/camper/day) is suggested.

Churches--A field study which provided information concerning church wastewater was conducted as part of a commercial water use study at Johns Hopkins University (Wolff, Linaweaver and Geyer, 1966). Quarterly billing records were obtained for at least a three-year period and recording water meters were installed for approximately one month at each of two churches. Annual water use, peak water demands and daily hydrographs were determined. As shown in Table A-51, the average annual per member usage was 0.52 L/day (0.14 gal/day) with a range of 0.04 to 0.94 L/day (0.01 to 0.25 gal/day). Daily water use measurements yielded maximum day and peak hour demands of 3.26 and 17.8 L/member/day (0.86 and 4.7 gal/member/day), respectively.

The daily hydrograph determined for the maximum water use day at one of the churches illustrates the effect of evening activities on water use (Figure A-17). Typical patterns of water use during days on which services alone are held are much lower, and do not reflect the magnitude of hourly use shown on this hydrograph.

TABLE A-51. CHURCH WATER USE CHARACTERISTICS
(After Wolff, Linaweaver and Geyer, 1966)

Parameter	Unit	Measurement	
		Annual Use	Daily Use
Mean Daily	L/day/member	0.52	0.74
Minimum Daily	L/day/member	0.04	-
Maximum Daily	L/day/member	0.94	3.26
Max. Qtr./Average Annual	-	1.29	-
Peak Hour	L/day/member	-	17.8

The suggested flows for churches are quite consistent around 11 to 19 L/seat/day (3 to 5 gal/seat/day) for churches without kitchen facilities and 19 to 30 L/seat/day (5 to 8 gal/seat/day) for those with kitchen facilities. For the above, the BOD_5 varies from 4.5 to 13.5 g/seat/day (0.01 to 0.03 lbs/seat/day). These values appear to be conservative and probably represent maximum conditions.

Country (golf) clubs--At Johns Hopkins University, the water use characteristics of golf clubs were also investigated (Wolff, Linaweaver and Geyer, 1966). Quarterly billing records were obtained for at least a three-year period and recording water meters were installed for approximately two weeks at each of the two country clubs. The mean annual usage per member for the two clubs studied was 250 L/day (66 gal/day) with a range of 168 to 305 L/day (44.5 to 80.7 gal/day). In both cases sprinkling demand was not a factor because sources other than the possible water supply were used. A hydrograph

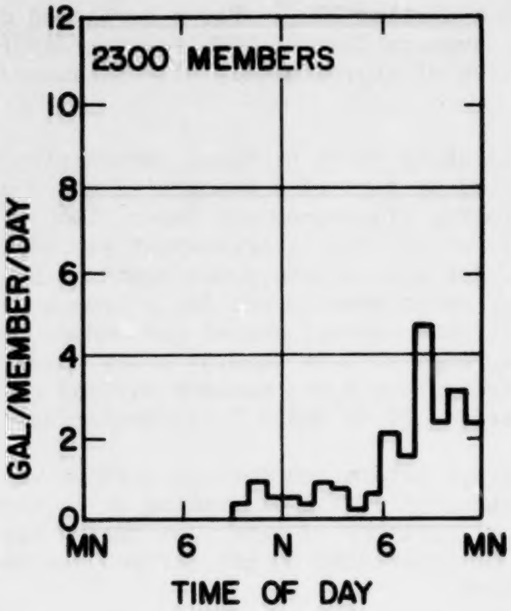


Figure A-17. Daily water use hydrograph for a church (Wolff, Linaweaver and Geyer, 1966).

for the maximum day recorded at one of the clubs is shown in Figure A-18. Two peaks are indicated, one before noon and one between 5 and 6 p.m.

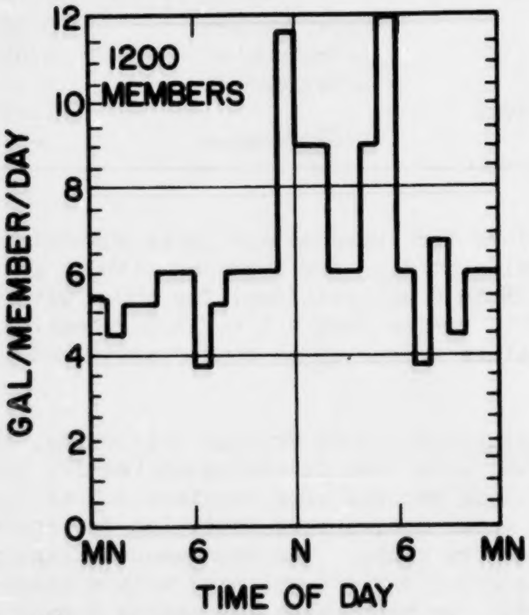


Figure A-18. Daily water hydrograph for a golf club (Wolff, Linaweaver and Geyer, 1966).

Typical guideline estimates for country club water use/waste production differentiate between transient members and resident members. The typical suggested flow is 95 L/day (25 gal/day) per transient member and 380 L/day (100 gal/day) for a resident member, with a typical BOD_5 contribution of 32 g/day (0.07 lb/day) and 90 g/day (0.20 lb/day), respectively.

Laundromats--Of particular interest are the daily wastewater flow volumes and usage patterns, as qualitative characteristics may be estimated using the results presented for household clotheswasher wastewater (Household Wastewater Characteristics section of this Appendix). Several field studies have been conducted to determine the quantitative characteristics of laundromat wastewater. At Manhattan College, while investigating synthetic detergent removal from laundry wastes, Eckenfelder and Barnhart (1962) obtained operating data from several laundromats. A typical installation sampled, was found to contain 30 washing machines, each of which used 113 L (30 gal) of water during its half-hour operating cycle. Under normal operating conditions, the authors reported there were 50 cycles/week/machine producing a total weekly wastewater volume of 5670 L (1500 gal).

The Oakland County Department of Public Works, Michigan, obtained water use data for ten laundromats from approximately one year of quarterly billing records prior to 1960 (Shah, 1976). The information generated is shown in Table A-52.

TABLE A-52. LAUNDROMAT WATER USE (Shah, 1976)

Laundromat	No. of Clotheswashers	L/day/Clotheswasher
1	20	990
2	20	1150
3	10	590
4	43	810
5	39	560
6	18	370
7	23	310
8	32	690
9	23	460
10	20	510

Based on the results of the survey, the average water use per washer was calculated to be 640 L/day (170 gal/day) with a range of 310 to 1150 L/day (83 to 304 gal/day).

In 1963, Flynn and Andres (1963) reported the results of research sponsored by the New York Water Pollution Control Board investigating the pollutional load of laundromat wastewater. The authors found that the average wastewater flow per machine varied from 340 to 910 L/day (89 to 240 gal/day), with the maximum flow for any laundromat studied equal to 2220 L/day (590 gal/day). The authors concluded that the minimum design flow for a laundromat should be based upon the manufacturer's stated capacity for the type of washer employed and an assumed 12-hour day with continual operation of all machines. For a 150 L (40 gal) washer operating on a 45-minute cycle, the design flow would be 2420 L/day/washer (640 gal/day/washer).

In the water use studies conducted at Johns Hopkins University (Wolff, Linaweaver and Geyer, 1966) the water use characteristics of coin-operated laundromats were investigated. Quarterly billing records of five laundromats were obtained for at least a three-year period and analyzed. The mean annual use was found to be 88 L/day per m^2 of establishment floor space (2.2 gal/day/ft^2) with a range of 31 to 270 L/day per m^2 (0.8 to 6.5 gal/day/ft^2). The upper limit of the 95% confidence interval was 260 L/day per m^2 (6.4 gal/day/ft^2). A recording water meter was installed at one of the five laundromats to determine a daily hydrograph. As shown in Figure A-19, water usage was prevalent and relatively constant between 7 a.m. and 4 p.m.

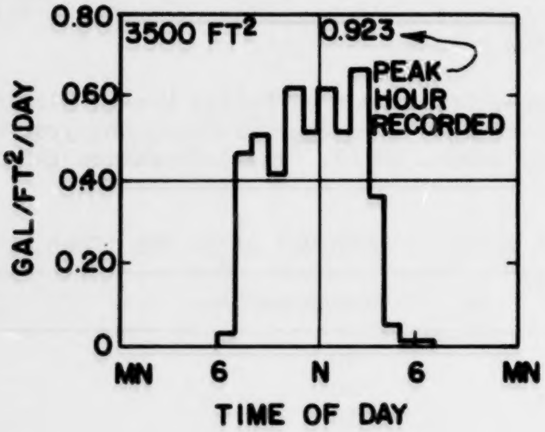


Figure A-19. Daily water use hydrograph for a laundromat (Wolff, Linaweaver and Geyer, 1966).

Based upon meter readings and extensive experience, the Washington Suburban Sanitary Commission (WSSC) determined that a typical laundromat can be expected to use an average of 150 L/day per m^2 (3.7 gal/day/ft^2) of establishment floor space (Bishop, 1975).

In Connecticut a laundromat was experiencing considerable difficulty and flow volumes were measured as part of a compliance order issued by the state (May, 1975). The average flow measured was 18,900 L/day (5000 gal/day) with a recorded maximum of 30,240 L/day (8000 gal/day). On a per machine basis, the average flow was about 450 L/day (120 gal/day) with the recorded maximum equal to 720 L/day (190 gal/day).

To facilitate comparison of the characteristics determined by each of the previously discussed studies, Table A-53 has been prepared. When the results of these studies are compared to the values listed in various general guideline tables, it can be seen that the average guideline estimates for flow are somewhat conservative, as they are typically in the range of 1500 to 1900 L/day/machine (400 to 500 gal/day/machine).

TABLE A-53. LAUNDROMAT WASTEWATER FLOW COMPARISON

Source	Unit	Mean	Range
Eckenfelder and Barnhart (1962)	L/machine/week	5670	-
Oakland County Department of Public Works (Shah, 1976)	L/machine/day	640	310-1150
Flynn and Andres (1963)	L/machine/day	-	340-910
Johns Hopkins University (Wolff, Linaweaver and Geyer, 1966)	L/m^2/day	88	2.9-24.7
Washington Suburban Sanitary Commission (Bishop, 1975)	L/m^2/day	150	-
Connecticut (May, 1975)	L/machine/day	450	? - 720

Marinas--The wastewater generated by a marina complex as a whole has not been the subject of any field studies to date and guidelines for the category "marina" are for the most part, non-existent. This is most likely due to the fact that the term "marina" describes a complex which includes smaller individual establishment units, such as a comfort station, restaurant, motel, and service station, and guidelines and studies (if performed) have been concerned with these smaller units. Thus, the marina category will not be considered further in this study. Pumpout wastes from the boats within a marina docking area are of separate concern and considerable data regarding their characteristics may be found elsewhere (Clark, 1968; Robbins and Green, 1974; Glueckert and Saigh, 1975).

Motels--Several investigations have been conducted which have provided information to estimate the characteristics of motel wastewaters. In 1960, Searcy and Furman (1961) studied water use by various institutions, including motels. Monthly rates were obtained from city records for an 18-month period; daily readings were taken directly from the motel water meters for at least a two-week period; and hourly readings were taken on several selected days. Six motels were selected for study. Two had restaurants which were served by separate meters, another had a coffee shop which served breakfast only, and three had no restaurant facilities. Based upon the results of the study, the authors suggested the following design flow rates:

24-hour average - 265 L/day/bed space
12-hour maximum - 400 L/day/bed space
4-hour maximum - 660 L/day/bed space
1-hour maximum - 1190 L/day/bed space

The daily flow pattern for a typical motel may be found in Figure A-20.

In 1962, Hubbell reported the results of a water consumption survey of 11 motels containing a total of 144 housing units. The average water use was found to be 350 L/day (92 gal/day) per housing unit. The author noted that in

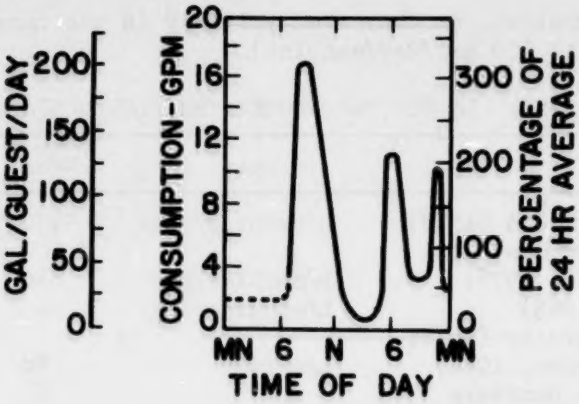


Figure A-20. Motel daily water use pattern (Searcy and Furman, 1961).

many cases, the average included owner use, sprinkling and miscellaneous use. Allowing for these losses, but compensating for low vacancy periods, Hubbell recommended a design load of 280 to 380 L/day (75 to 100 gal/day) of domestic strength wastewater per occupied unit.

As part of the water use studies conducted at Johns Hopkins University in the early 1960's, motel water use characteristics were determined. As part of the residential water use study (Linaweaver, Geyer and Wolff, 1967), a single motel was monitored and the following characteristics were measured:

Units - 166
Average Annual Demand - 260 L/day/unit
Maximum Recorded Day - 370 L/day/unit
Maximum Hour - 490 L/day/unit
Peak Hour - 7 a.m. - 8 a.m.

A typical daily water use hydrograph for the motel studied is shown in Figure A-21.

Water use data from five motels were collected and analyzed as part of the Johns Hopkins commercial water use study (Wolff, Linaweaver and Geyer, 1966). Based upon the inspection of three years of quarterly billing records, the mean annual water use for the motels studied was found to be 9 L/day per m^2 (0.22 gal/day/ft^2) of floor space with a range of usage varying from 6.3 to 11 L/day/m^2 (0.16 to 0.27 gal/day/ft^2). Assuming 18 m^2 (190 ft^2) per housing unit, these figures would convert to 170, 330 and 580 L/day/unit (45, 31 and 54 gal/day/unit), respectively. The maximum demand was found to occur during the summer quarter, since a number of the motels contained swimming pools. Nearly all of the motels had restaurants.

Recording water meters were installed at three of the motels for a total of 117 days (of which 40 generated useable data). Recorded maximum daily

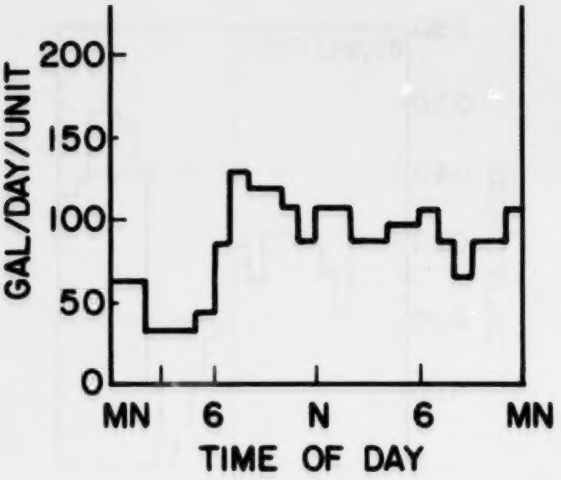


Figure A-21. Daily water use pattern for a motel (Linaweaver, Geyer and Wolff, 1967).

usage was 11.3 L/day/m^2 (0.28 gal/day/ft^2). The ratio of the maximum day to the daily mean was 2.06 with the maximum hour to the mean, 6.92. A daily hydrograph for one of the motels studied is shown in Figure A-22. Two large peaks were found to occur, between 8 and 10 a.m. and 8 and 10 p.m. It should be noted that the motels studied contained facilities (restaurant, bar and/or swimming pool) whose water use were included in the measured water use rates. Also, the provision of these types of facilities along with the housing units, makes the motels studied more sophisticated than those providing only rooms and vending machines. However, the water use rates may be applicable to smaller, simpler motels since the total floor area of the motel complex was used to compute the flow rates.

Several local governmental units have provided additional information on motel wastewater. The sewer design section of the County Sanitation Districts of Los Angeles County has developed information on the quantity of sewage generated by various establishments, including motels (Fuller, 1975). An average flow of 230 L/customer/day (62 gal/customer/day) with a peak flow of 680 L/customer/day (181 gal/customer/day) is used by Los Angeles County.

The Washington Suburban Sanitary Commission reported the results of a motel water use study in 1968 (WSSC, 1968). Water use information was obtained for ten motels from the Water Registrar for the year, 1967. For six motels with attached restaurants the average usage per rental unit was found to be 660 L/day (175 gal/day) with a range of 450 to 880 L/day (119 to 234 gal/day) and a median of 680 L/day (181 gal/day). For four motels without restaurants, the average usage per rental unit was found to be 480 L/day (128 gal/day), with a range of 420 to 550 L/day (111 to 145 gal/day) and a median of 470 L/day (125 gal/day).

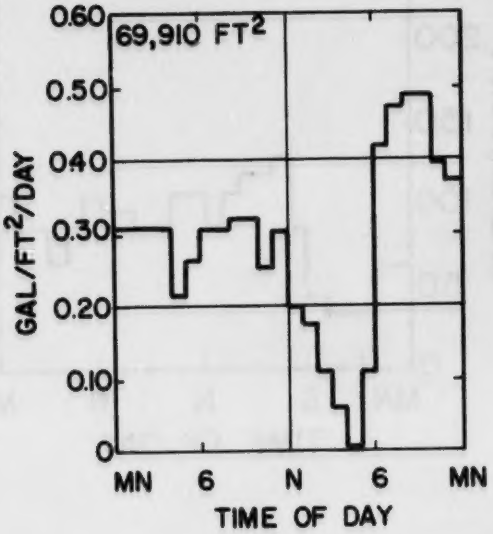


Figure A-22. Motel water use pattern (Wolff, Linaweaver and Geyer, 1966).

In 1976, the Oakland County Department of Public Works in Michigan reported the results of a recent study of motel water consumption (Shah, 1976). Water use records for a 3-year period at 26 motels in Oakland County were analyzed. It was discovered that motels with less than 50 rooms had water use characteristics distinctly different from motels with 50 rooms or above. Most of the motels with 50 rooms or above were found to have restaurants and recreation facilities which contributed to a higher usage. The results of the study are summarized in Table A-54.

TABLE A-54. MOTEL WATER USE (Shah, 1976)

Motel Size	Number of Motels	Average No. of Rooms	L/day/room	
			Average	Range
Below 50 rooms	13	28	400	72 - 720
Above 50 rooms	13	183	710	360 - 1360

To facilitate comparison of the flow values determined in each study, Table A-55 has been prepared. Typical guidelines for flow compare reasonably well with the average values shown in Table A-55, as the recommended flow is commonly 150 to 190 L/day/bed space (40 to 50 gal/day/bed space) or 380 L/day/unit (100 gal/day/unit).

TABLE A-55. MOTEL WASTEWATER FLOW COMPARISON

Study	Flow
Searcy and Furman (1961)	265 L/day per bed space
Hubbell (1962)	350 L/day per housing unit
Linaweaver, Geyer & Wolff (1967)	260 L/day per unit
Wolff, Linaweaver & Geyer (1966)	9 L/day per m^2
County Sanitation Districts of Los Angeles County (Fuller, 1975)	230 L/day per customer
Washington Suburban Sanitary Commission (WSSC, 1968)	480 L - 660 L/day per rental unit
Oakland County DPW (Shah, 1976)	400 L/day per rental unit (< 50 room motels) 710 L/day per rental unit (> 50 room motels)

Restaurants--The characteristics of restaurant wastewater have been studied in some detail by various investigators. Restaurants can be divided into two categories, conventional sit-downs and drive-ins. A separate discussion of each follows.

Searcy and Furman (1961) studied the water use of two restaurants and one cafeteria. The restaurants were operated in conjunction with motels and the cafeteria was a part of a shopping center. Three meals per day were served by the restaurants and two were served by the cafeteria. Based on quarterly billing records and daily meter readings, the average water use was measured as 24 to 29 L/meal/day (6.4 to 7.7 gal/meal/day). The daily flow pattern for a typical restaurant is shown in Figure A-23.

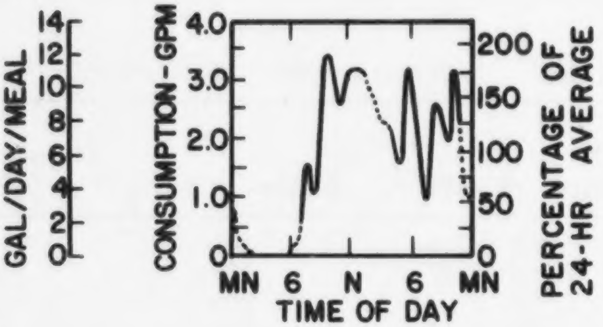


Figure A-23. Restaurant daily water use pattern (Searcy and Furman, 1961).

Based upon the results of their investigation, the authors recommended the following design flows:

24-hour average - 34 L/meal/day
12-hour maximum - 48 L/meal/day
4-hour maximum - 58 L/meal/day
1-hour maximum - 68 L/meal/day

Hubbell (1962) reported that a large restaurant in Birmingham, Michigan employing about 120 and containing 376 seats in a cafeteria and dining room, had a 1960 water consumption of 230 L/seat/working day (61 gal/seat/working day). This amounted to 44 L/meal (11.7 gal/meal). Complete garbage grinding facilities were used at the restaurant.

The water use characteristics of fourteen restaurants were evaluated at Johns Hopkins University during the early 1960's (Wolff, Linaweaver and Geyer, 1966). Based upon the inspection of three years of quarterly billing records, the mean annual water use was found to be 91 L/seat/day (24.2 gal/seat/day) with a range of usage varying from 9.5 to 260 L/seat/day (2.5 to 67.9 gal/seat/day). The upper limit of the 95% confidence interval was 210 L/seat/day (55.2 gal/seat/day). Use during the maximum quarter was 135 L/seat/day (35.4 gal/seat/day).

A recording water meter was installed at one of the restaurants for a total of 21 days (11 produced useable data). The average daily use was 230 L/seat/day (61 gal/seat/day). The maximum day during the monitoring amounted to 320 L/seat/day (83.4 gal/seat/day) with a peak hour of 630 L/seat/day (167 gal/seat/day). A daily hydrograph was developed and is shown in Figure A-24. The operation of a typical conventional restaurant was found to closely follow the water use hydrograph of a residence. A peak was reached in the forenoon, between 2 and 5 p.m. and between 6 and 8 in the evening. Another peak was experienced after midnight.

The State of Florida sponsored a water use study which included several cafeteria restaurants (Santarone, 1976). Monthly water meter readings were obtained at each of four establishments producing the results shown in Table A-56.

TABLE A-56. CAFETERIA WATER USE (Santarone, 1976)

			L/seat/day	
Cafeteria	Months of Data	Number of Seats	Average	Range
A	17	226	95	15-140
B	23	200	240	150-390
C	24	200	120	72-160
D	12	424	91	64-110

The City of Honolulu obtained composite samples from each of two restaurants over a five-day period as a part of a wastewater quality survey (Hayashida, 1975). The average characteristics determined are shown in Table A-57.

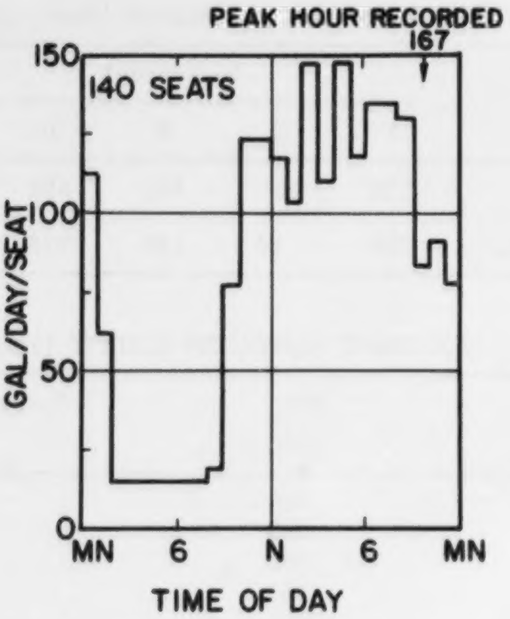


Figure A-24. Restaurant daily water use pattern (Wolff, Linaweaver and Geyer, 1966).

TABLE A-57. RESTAURANT WASTEWATER CHARACTERISTICS (Hayashida, 1975)

Parameter	Restaurant #1	Restaurant #2	Average
BOD, mg/L	759	525	640
COD, mg/L	3606	1367	2500
Suspended Solids, mg/L	800	202	500
Grease, mg/L	663	356	510
pH	6.8-8.2	8.9-11.3	-

The County Sanitation District of Los Angeles County uses 31 L/customer/day (8.1 gal/customer/day) as the average restaurant flow with 64 L/customer/day (17 gal/customer/day) as the peak flow (Fuller, 1975).

To determine if waste surcharges were necessary, the City of Greensboro, North Carolina collected samples from each of five restaurants (Shaw, 1970). Twenty-four hour, flow composited samples were obtained from each facility over a three-day period and average characteristics were determined. The results are shown in Table A-58.

The Philadelphia Water Department obtained several twenty-four hour composite samples from local restaurants and the results of qualitative analyses yielded the results shown in Table A-59.

TABLE A-58. RESTAURANT WASTEWATER (Shaw, 1970)

Parameter	Restaurant A	B	C	D	E	Average
BOD, mg/L	531	390	423	651	737	546
Suspended Solids, mg/L	286	48	172	378	402	257

TABLE A-59. RESTAURANT WASTEWATER QUALITY (Kulesza, 1975)

Restaurant	BOD_5 mg/L	Suspended Solids mg/L
A	960	1428
A	630	828
A	880	1058
B	280	172
C	750	1112
C	750	846
D	610	1200
D	570	918
E	680	1985
E	435	728
Average	655	1030

To facilitate comparison of the results of the preceding field studies, Table A-60 has been prepared with the average values for several parameters shown. Most of the general guidelines tables include restaurants, and the typical flow given is 38 L/patron/day (10 gal/patron/day) with a BOD_5 loading of 9.1 g/patron/day (0.02 lbs/patron/day) or 250 mg/L. On a per seat basis, the typical flow is 130 L/day (35 gal/day). When compared to the results of the field studies shown in Table A-60, the guideline flows appear appropriate although the estimated BOD_5 contribution is somewhat low.

A single study was identified which investigated the water use of drive-in restaurants, as part of the commercial water use study at Johns Hopkins University (Wolff, Linaweaver and Geyer, 1966). The authors identified two types of drive-in restaurants, those with seating facilities and those which have little or no seating facilities. Based upon quarterly billing records of a drive-in with little seating, an average water use of 410 L/day (109 gal/day) per car space was identified with the ratio of the maximum quarter to the average annual use equal to 1.78. The authors recommended an average design water use of 380 L/day (100 gal/day) per car space for this type of establishment.

TABLE A-60. RESTAURANT WASTEWATER COMPARISON

	Parameter			
Study	BOD_5 mg/L	Suspended Solids mg/L	Grease mg/L	Flow L/day per unit
Searcy and Furman (1961)	-	-	-	34 per meal
Hubbell (1962)	-	-	-	44 per meal
Johns Hopkins (Wolff, Linaweaver and Geyer, 1966)	-	-	-	91 per seat
Florida (Santarone, 1976)	-	-	-	91-240 per seat
Honolulu, Hawaii (Hayashida, 1975)	640	500	510	-
Los Angeles County, California (May, 1975)	-	-	-	31 per patron
Greensboro, North Carolina (Shaw, 1970)	546	257	-	-
Philadelphia, Pennsylvania (Kulesza, 1975)	655	1030	-	-

Three typical drive-ins having seating arrangements were also studied. The mean annual water use was found to be 150 L/day (41 gal/day) per seat with a range of usage of 140 to 180 L/day (37.8 to 46.4 gal/day). A recording water meter was installed at one of the drive-ins and a daily water use hydrograph was developed as shown in Figure A-25. The hydrograph is for a typical summer day.

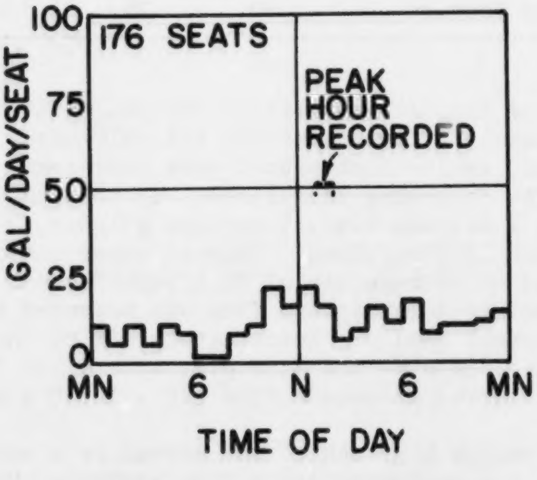


Figure A-25. Drive-in restaurant water use pattern (Wolff, Linaweaver and Geyer, 1966).

Most of the general guidelines do not differentiate between conventional sit-down restaurants and drive-ins. Those that do, indicate an average flow of 7.6 to 15.2 L/patron/day (2 to 4 gal/patron/day) or 190 L/day/car space (50 gal/day/car space).

Schools--Considerable effort has been devoted to monitoring the water use and wastewater production in public and private schools. Wisnieski and Garber (1953) studied the annual water consumption and student population of several schools for a five-year period. The results of this study have been summarized in Table A-61. The authors noted certain cautions which should be observed when applying the results of their study. If a school is to be used for numerous evening meetings, special courses and athletic activities, the quantities must be increased accordingly; several such schools have reported water use from 76 to 152 L/cap/day (20 to 40 gal/cap/day). However, the occasional use of the school building for PTA meetings, Boy Scout meetings, and dances should not increase the per capita water use significantly.

TABLE A-61. SCHOOL WATER USE - L/cap/day
(Wisnieski and Garber, 1953)

School Type	Facilities Provided	No. of Schools	Mean	Range	90% Probability
Grade Schools	No Cafeteria or Showers	31	23	9.1 - 55	39
Grade Schools	Cafeterias	10	23	9.8 - 42	40
Junior High Schools	With or Without Cafeteria and Showers	16	21	8.7 - 34	37
High Schools	With Cafeteria and Showers	30	34	9.8 - 79	59

In planning waste disposal facilities for two existing schools, Coberly (1957) measured the sewage flows to obtain reliable data. A grade school and combination junior and senior high school were monitored. The grade school had 437 pupils and the following facilities: 13 toilets, 3 urinals, 2 service sinks, 1 dishwasher, 1 kitchen sink, 1 garbage grinder, 2 wash fountains, 10 drinking fountains and 13 classrooms. Metered water use over a two-month Fall period indicated an average use of 21 L/pupil/school day (5.6 gal/pupil/school day). The average daily sewage flow was measured as 17 L/pupil/school day (4.4 gal/pupil/school day) or approximately 80% of the water used. The flow occurred between 7:30 a.m. and 5:30 p.m. with minor flows after those hours. Peak flows occurred at recess time (10 - 10:30 a.m.), noon and 2 p.m.

The junior and senior high school was served by a sewer line which also served a greenhouse, one residence and a fire station. Making allowances for these, the authors determined an average flow of 21,000 L/day (5549 gal/day) with an attendance of 1430 pupils, which represented 15 L/pupil/school day (3.9 gal/pupil/school day). A broad peak was found to occur about 5:30 p.m. and was probably due to showering after athletic activities.

Searcy and Furman (1961) studied school water consumption extensively at nine elementary schools, two junior high schools, two senior high schools and one combined school (kindergarten through twelfth grade). Monthly water use rates were obtained from city records for an 18-month period; daily readings were taken directly from the school water meters for at least a two-week period; and hourly readings were taken on several selected days. All elementary schools were equipped with cafeterias while the junior and senior high schools and the combined school included a cafeteria and a gymnasium with showers. Although water use was determined per pupil per day, the authors observed that there was not always a direct relationship between water use and students in attendance. Variations in water use appeared to be more nearly related to special activities at the school. Nevertheless, since the number of students did indicate school size, it was used as the basis for expressing water use.

The average daily water use determined, based upon daily and monthly meter readings is shown in Table A-62. The daily hydrographs developed for each type of school based on hourly meter readings are presented in Figure A-26. It was noted that over 82% of the water used each day was consumed in a 10-14 hour period. In the elementary and junior high schools, 93% of the total daily flow occurred within a 10-hour period. The effect of showers following the sports program explained the second peak in the consumption pattern of the high schools and the combined schools.

TABLE A-62. SCHOOL WATER USE, L/cap/day
(Searcy and Furman, 1961)

Type of School	Daily Readings Mean	Daily Readings Range	Monthly Records Mean	Monthly Records Range
Average Elementary	28	21 - 40	22	0.8 - 49
Average Junior High	25	15 - 46	23	0.4 - 61
Average Senior High	75	34 - 230	44	1.5 - 140
Combined	51	46 - 56	45	7.2 - 70

Based on a review of all data collected during their study, the investigators recommended the design flows presented in Table A-63.

TABLE A-63. SCHOOL WATER USE, L/student/day
(Searcy and Furman, 1961)

Type of School	Suggested Design Flows 24-Hour Average	12-Hour Maximum	4-Hour Maximum	1-Hour Maximum
Elementary	38	76	150	190
Junior High	45	91	180	230
Senior High	76	130	190	250
Combined	53	93	130	170

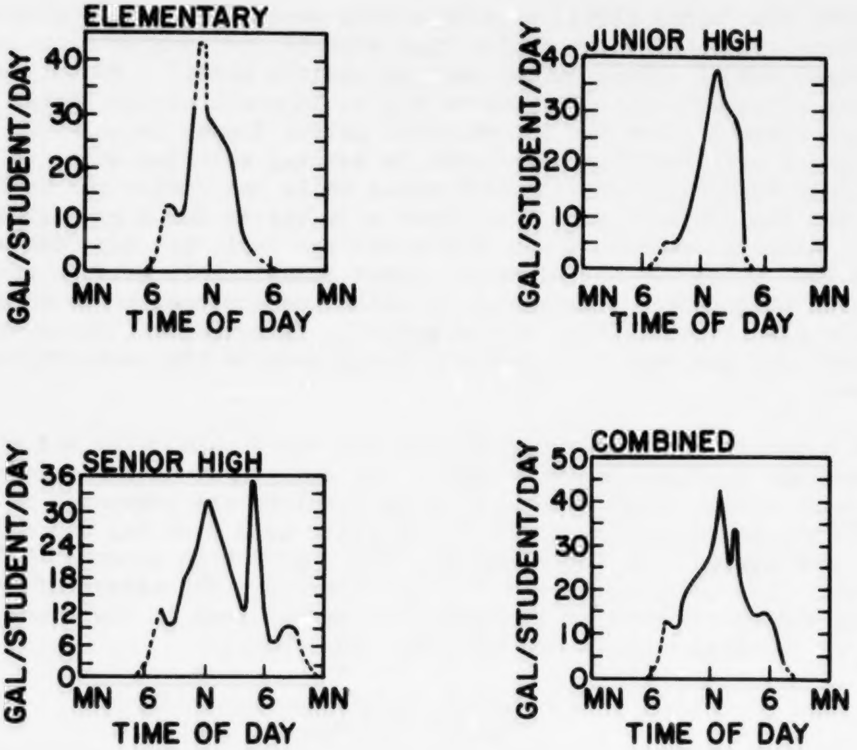


Figure A-26. School water use hydrographs (Searcy and Furman, 1961).

Hubbell (1962) reported that day schools without shower or cafeteria facilities generally produce a daily per capita load of 38 L (10 gal) and 13.5 g (0.03 lbs) of BOD_5, primarily from toilet wastes. Shower and cafeteria facilities were cited as potentially adding an additional 38 L (10 gal) and 9 g (0.02 lbs) of BOD_5 to the above values.

Reeder and Fogarty (1964) actually metered sewage output from schools while simultaneously recording the water use at 19 schools in Dade County, Florida. All schools were serviced by cafeterias while only six had gymnasiums and shower facilities. Thirteen elementary and junior high schools without showers had between 277 and 1269 pupils while six junior and senior high schools with showers had between 830 and 2645 pupils. Five days of water use and sewage flow data were recorded simultaneously at each school. During each measurement period, the custodial personnel were instructed not to water lawns, wash windows or otherwise use water that would not be returned as wastewater. The average and various maximum rates of flow determined in this study are presented in Table A-64.

As part of the commercial water use study at Johns Hopkins University, the water use characteristics of elementary, junior and senior high schools in the public and private categories were determined (Wolff, Linaweaver and Geyer, TABLE A-64. SCHOOL WATER USE AND WASTEWATER FLOWS, L/cap/day (after Reeder and Fogarty, 1964)

	Schools With Restrooms & Cafeterias		Schools With Restrooms, Cafeteria & Showers	
Parameter	Water	Sewage	Water	Sewage
No. of Schools	13	13	6	6
Days of Data	57	57	28	28
Mean of all Days	28	26	32	30
S.D. of all Days	5.2	5.2	5.7	5.7
Mean of Max. Days at all Schools	31	28	36	31
Maximum Day Observed	44	39	50	46
Mean of School with Max. Day	36	37	42	36
Mean of 5-hour Max. at all Schools	-	106	-	100

1966). Quarterly billing records were obtained for at least a three-year period and recording water meters were installed at several schools. The characteristics determined are shown in Table A-65.

TABLE A-65. WATER USE IN SCHOOLS - L/student/day (Wolff, Linaweaver and Geyer, 1966)

	Annual Records*				Daily Metering		
School Type	No. of Schools	Mean	Range	U.L. 95% C.I.	Mean	Max. Day	Peak Hour
Public							
Elementary	9	20	15 - 37	33	14	26	130
Junior High	3	21	10 - 36	37	-	-	-
Senior High	7	25	8.1 - 40	46	20	57	350
Private							
Elementary	5	8.6	1.5 - 19	23	11	12	97
Senior High	4	39	21 - 55	70	54	59	150
Combined (1-12)	5	32	7.4 - 64	70	32	64	190

* School year.

Graphs of daily water use patterns over an entire week are shown in Figure A-27 for an elementary school and a senior high school. The hydrographs indicate the greater magnitude of demands in the high school. As shown in the figure, the elementary school exhibits a decided peak at noon,

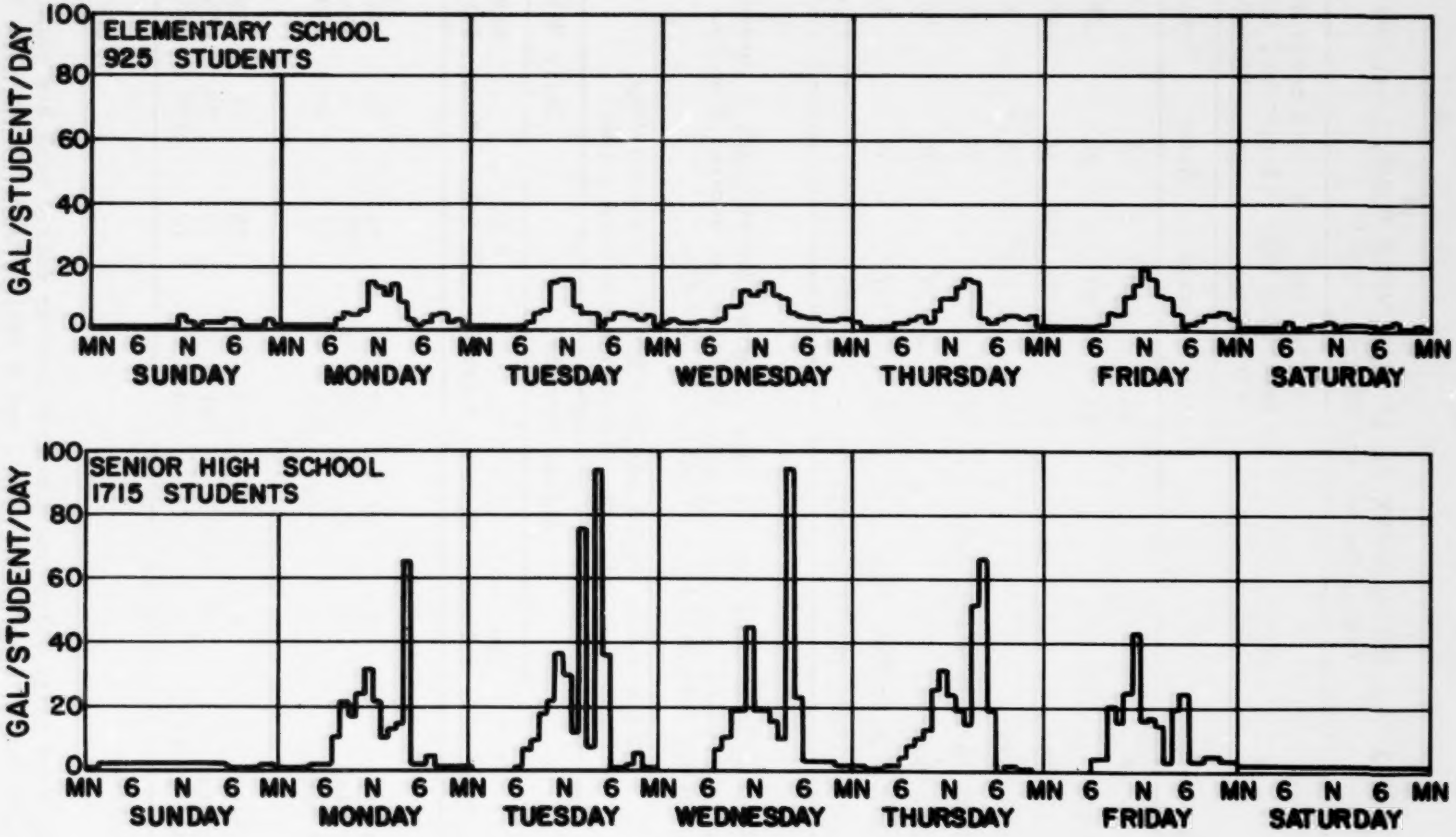


Figure A-27. Weekly flow pattern for an elementary and secondary school (Wolff, Linaweaver and Geyer, 1966).

while the high school experiences a decided double peak, one at or before noon and one late in the afternoon. The latter peak is substantially higher than the noon peak. The investigators noted that it was due to the athletic activities which are offered in high school but not in elementary schools.

The Oakland County (Michigan) Department of Public Works investigated the water consumption of 33 schools for two years (Ringler, 1975). The consumption rates shown in Table A-66 were determined.

TABLE A-66. SCHOOL WATER USE, L/student/day
(Ringler, 1975)

Type of School	No. of Schools	Mean	S.D.	Range
Elementary	7	33	22	10 - 74
Secondary/Other	26	31	24	5.7 - 130

The Iowa State Health Department analyzed the water use by several Des Moines schools utilizing records from the Des Moines School Administration office and the Des Moines Water Works (Evans, 1976). The average per capita use per school day determined for four types of schools is shown in Table A-67.

TABLE A-67. AVERAGE SCHOOL WATER USE
(Evans, 1976)

Type of School	Facilities Provided	No. of Schools	Average Enrollment	L/student/day	
				Average	Range
Grade School	No Hot Lunch No Showers	9	400	19	12 - 28
Grade School	Hot Lunch No Showers	5	500	25	9.1 - 41
Junior High School	Hot Lunch Showers	3	716	52	51 - 53
Senior High School	Hot Lunch Showers Swimming Pools	4	1680	61	54 - 68

To facilitate comparison of the flow information determined in each study, the average flows determined by each are listed in Table A-68. As shown, there is fair agreement in the results of the individual studies.

Most of the general guidelines include three classes of schools, based on the facilities provided; (1) schools with restrooms only, (2) schools with restrooms and cafeteria, and (3) schools with restrooms, cafeteria and showers.

TABLE A-68. AVERAGE SCHOOL WASTEWATER FLOW COMPARISON, L/student/day

Study	Elementary School	Junior High School	Senior High School
Wisnieski and Garber (1953)	23	21	34
Coberly (1957)	17	15	-
Searcy and Furman (1961)	38	45	76
Reeder and Fogarty (1964)	28	28	32
Johns Hopkins University (Wolff, Linaweaver and Geyer, 1966)	14	-	20
Oakland County, Michigan (Ringler, 1976)	33	31	31
Iowa (Evans, 1976)	19	52	61

The suggested flows for these three classes are typically 57, 76 and 94 L/student/day (15, 20 and 25 gal/student/day), respectively. When compared to the results of the individual studies, these guideline flows are very conservative for average flows and even somewhat conservative for peak flows.

Service stations--As a part of a residential water use study, a single service station was monitored by Linaweaver, Geyer & Wolff (1967), and the following water use characteristics were measured:

Lifts - 1
Average Annual Flow - 1780 L/day
Maximum Day - 6620 L/day
Maximum Hour - 47,250 L/day (6 p.m.-7 p.m.)

As part of the commercial water study at Johns Hopkins University, water use data were obtained from 6 service stations (Wolff, Linaweaver and Geyer, 1966). All six had a small office and an attached garage, having one or two car lifts, and storage space for 2 or 3 cars. Based upon the inspection of three years of quarterly billing records, the mean annual water use was found to be 10.2 L/day/m^2 (0.25 gal/day/ft^2) of combined garage and office floor space. The range of usage was 6.5 to 18 L/day/m^2 (0.16 to 0.44 gal/day/ft^2), with the maximum usage occurring during the summer.

Recording water meters were installed at three of the service stations for a total of 65 days (31 produced useable data). The mean daily usage measured was 7.2 L/day/m^2 (0.18 gal/day/ft^2). The maximum day recorded was 17 L/day/m^2 (0.41 gal/day/ft^2), while the peak hour was 140 L/day/m^2 (3.4 gal/day/ft^2). A daily hydrograph was determined for one of the service stations and indicated a peak usage during the early afternoon hours (Figure A-28).

General guideline estimates for service station flows are typically expressed in terms of vehicles served or the number of pump islands or service bays. Suggested flows are 38 L/day (10 gal/day) per vehicle served or 1900 to 3800 L/day (500 to 1000 gal/day) per island or bay present.

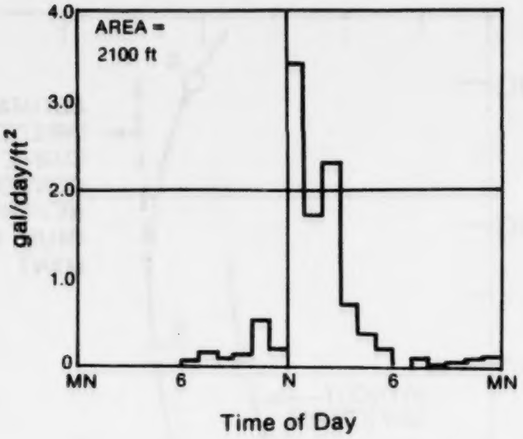


Figure A-28. Service station water use pattern (Wolff, Linaweaver and Geyer, 1966).

Shopping centers--A survey conducted in 1961, revealed information on the water use characteristics of shopping centers (Anonymous, 1961). In one study, monthly meter readings on each business establishment within six shopping centers in the Miami area were obtained. The average consumption varied from 5.82 to 8.54 L/day per m^2 of store floor space (0.143 to 0.210 gal/day/ft^2), while the maximum monthly consumption values ranged from 7.16 to 12.45 L/day/m^2 (0.176 to 0.306 gal/day/ft^2). The data compiled was graphically presented as shown in Figure A-29. In a second study, three shopping centers near Springfield, Massachusetts were metered for one year yielding the following water use values:

Shopping center A (1490 m^2) - 91 L/day/m^2
Shopping center B (4460 m^2) 1.26 L/day/m^2
Shopping center C (21,300 m^2) - 2.97 L/day/m^2

The high usage in shopping center A was caused by a laundromat. It should be noted that the shopping centers studied in the Miama area did not contain laundromats.

Searcy and Furman (1961) studied water use at a single shopping center. Monthly rates were obtained from city records for an 18-month period; daily readings were taken directly from the component establishment water meters for at least a two-week period; and hourly readings were taken on several selected days. The results of the study are shown in Table A-69 and Figure A-30.

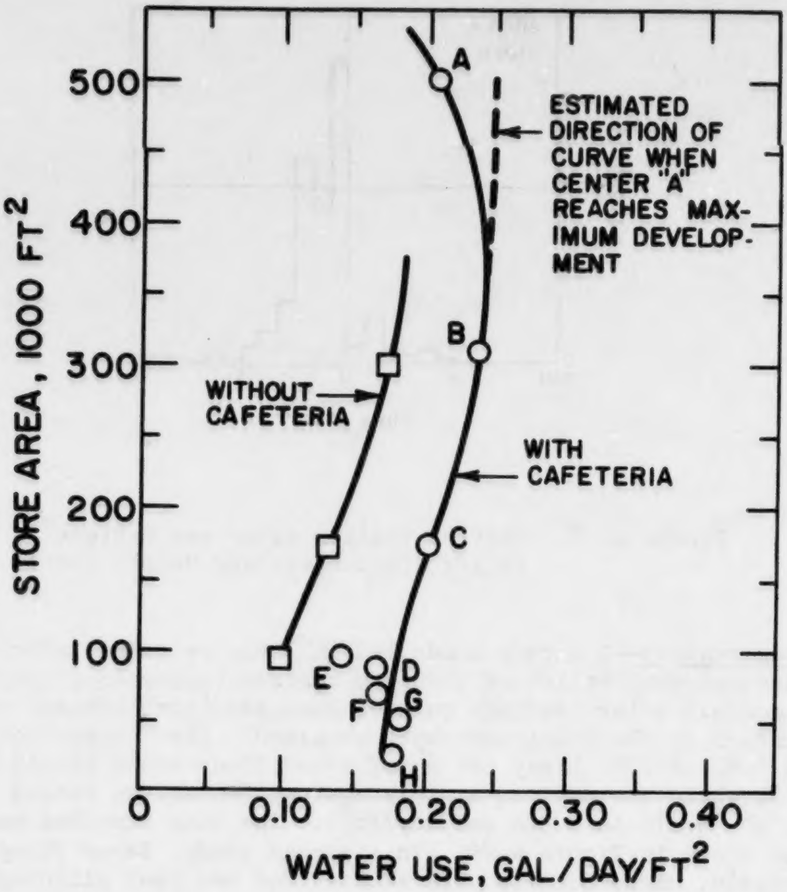


Figure A-29. Shopping center water use (Anonymous, 1961).

TABLE A-69. SHOPPING CENTER WATER USE, L/day/m^2 (Searcy and Furman, 1961)

Measurement Period	Average	Range
Monthly Readings	8.50	5.29 - 11.3
Daily Readings	7.12	3.34 - 8.83
Hourly Readings	8.83	? - 18.3

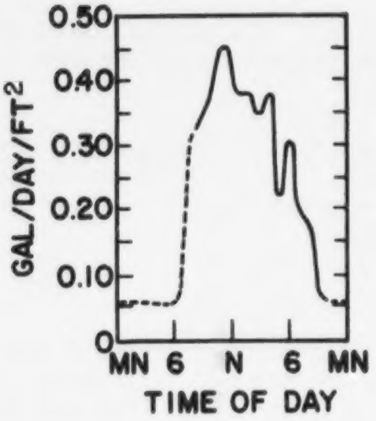


Figure A-30. Water consumption pattern of a shopping center (Searcy and Furman, 1961).

Based upon the results of the study, the investigators recommended the following design flows:

24-Hour Average - 10.2 L/day/m^2
12-Hour Maximum - 15.3 L/day/m^2
4-Hour Maximum - 19.3 L/day/m^2
1-Hour Maximum - 21.4 L/day/m^2

Hubbell (1962) reported that large shopping centers usually contain a department store and a variety of smaller establishments and employ from 80 to 140 equivalent 8-hour employees per acre of building area. Restaurants, supermarkets, laundromats and car washes are usually among the primary sources of wastewater, while most other establishments contribute nominal amounts of wastewater, primarily from employee and public restrooms. Average water use and estimated wastewater production from three large shopping centers in the Detroit area as reported by Hubbell are presented in Table A-70.

TABLE A-70. SHOPPING CENTER WATER USE AND WASTEWATER PRODUCTION (Hubbell, 1962)

Shopping Center	No. of Employees	Building Area (m^2)	Water Use L/day/m^2	Wastewater Produced L/day/m^2
A	1500	58,300	6.51	5.37
B	4500	116,200	10.8	8.87
C	400	98,400	10.7	8.79

A water use survey at Center A on a typical Saturday indicated the following ratios between the average water consumption rate and the rate during the indicated period:

Measurement Period	Measurement Period Rate / 24-Hour Average Rate
15 Minute	2.54
1 Hour	2.03
6 Hour	1.83

Investigators at Johns Hopkins University studied the water use characteristics of two shopping centers and their component stores producing the results shown in Table A-71 (Linaweaver, Geyer and Wolff, 1967).

TABLE A-71. SHOPPING CENTER WATER USE, L/day/m^2
(Linaweaver, Geyer and Wolff, 1967)

Shopping Center	Building Area	Mean Annual Use	Maximum Day	Maximum Hour
A	22,300 m^2	6.02	8.46	15.3 (2 p.m. - 3 p.m.)
B	13,500 m^2	7.28	-	-

A daily hydrograph developed for a typical day at Center A is shown in Figure A-31.

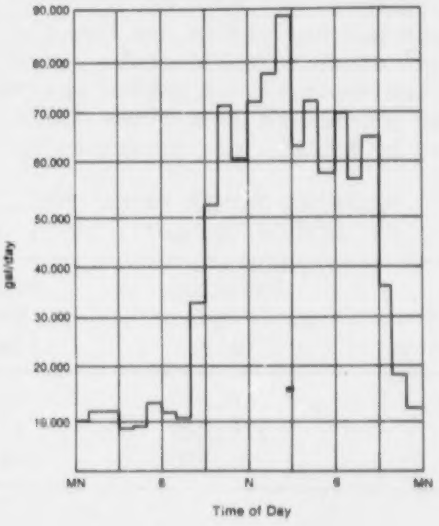


Figure A-31. Daily water use pattern for a shopping center (Linaweaver, Geyer and Wolff, 1967).

A second water use study was conducted at Johns Hopkins University and data for five large department stores were analyzed (Wolff, Linaweaver and Geyer, 1966). Department stores are often the major component of a shopping center and therefore, merit discussion. Based on the inspection of three years of quarterly billing records, the mean annual water use was 8.79 L/day per m^2 of total sales area (0.216 gal/day/ft^2) with a range of 4.47 to 16.4 L/day/m^2 (0.11 to 0.40 gal/day/ft^2).

Recording water meters were installed at two of the stores for a total of 110 days (42 produced useable data). Recorded maximum daily usage was 15.8 L/day/m^2 (0.388 gal/day/ft^2) and the peak hour was 40.0 L/day/m^2 (0.958 gal/day/ft^2). Hydrographs were developed as shown in Figure A-32 for each of two large department stores. The mean annual water usage of 1.75 L/day/m^2 (0.043 gal/day/ft^2) for Store A compared to 7.81 L/day/m^2 (0.192 gal/day/ft^2) for Store B reflects the diverse characteristics of the two establishments. Store A was a typical discount store offering minimal water using facilities, while Store B was a well appointed department store, offering a restaurant, large restrooms and other facilities.

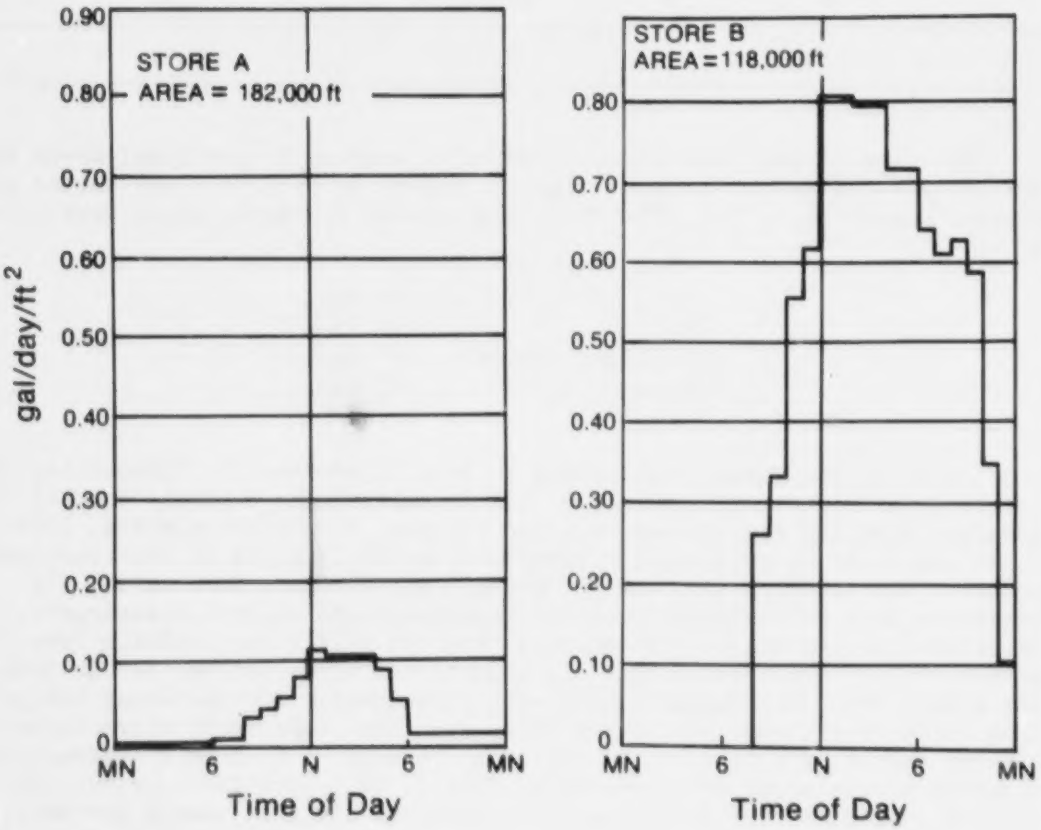


Figure A-32. Daily water use patterns for department stores (Wolff, Linaweaver and Geyer, 1966).

In 1974, Feherty reported the findings of recent studies on several shopping centers of varying size. The average daily water use related to total floor area for a number of shopping centers in Ontario is shown in Table A-72.

TABLE A-72. SHOPPING CENTER WATER USE, L/day/m^2 (Feherty, 1974)

Shopping Center	Floor Area (m^2)	Mean Water Use
A	3,900	21.6
B	20,900	11.0
C	7,400	11.0
D	52,000	8.13
E	109,400	8.13
F	74,300	7.32
G	39,000	4.47
H	44,800	4.47
I	4,700	3.25
J	14,900	2.03
K	10,700	1.63
Average	34,700	7.55

The City of Honolulu obtained composite samples from a local shopping center over a five-day period during the summer as part of a wastewater quality survey (Hayashida, 1975). The following average characteristics were determined:

BOD - 270 mg/L
COD - 1344 mg/L
Suspended Solids - 337 mg/L
Grease - 67 mg/L
pH - 7.2-7.8

In 1975, the results of a study to develop methods for forecasting water use by department stores and commercial establishments commonly located in suburban shopping centers was reported (McCuen, Sutherland and Kim, 1975). Water use data was obtained for commercial establishments in four shopping centers, one in Wisconsin, one in Maryland and two near Washington, D.C. Water-use data was gathered from two department stores with restaurants, five department stores without restaurants, and 140 mall shops (falling into 24 categories). It was noted that air conditioner water use was not included in the data. Water use relationships were subsequently derived using the data from these establishments. The authors felt that department store water use was the result of activities of employees as well as customers. After considering a variety of proxy variables felt to be indicative of water use, the authors concluded that the gross store area was the most useful parameter. The water use data gathered for the department stores is shown in Table A-73. The data for the department stores without restaurants was analyzed using linear regression yielding the following water use relationship:

$$\text{Water Use, L/day} = -5866 + 2.260\ [\text{Gross store area in m}^2] \qquad (3)$$

This equation provided a correlation coefficient of 0.92 and a standard error or estimate equal to 5745 L/day, significantly less than the 12,320 L/day standard deviation for the water use data. Due to the insufficient data for department stores with restaurants, a similar analysis was not performed on that category.

TABLE A-73. DEPARTMENT STORE WATER USE (McCuen, Sutherland and Kim, 1975)

Parameter	Stores Without Restaurants	Stores With Restaurants
Number of Stores	5	2
Water Use, L/day		
Mean	24760	45960
Standard Deviation	12320	-
Range	6890 - 71180	34810 - 57120
Gross Store Area, m^2		
Mean	13550	15710
Standard Deviation	4990	-
Range	8180 - 20330	14780 - 16080
Mean Water Use/Store Area L/day/m^2	1.83	2.93

McCuen et al. found water use in mall shops to be considerably less than that of department stores, primarily due to the fact that the shop restroom facilities were usually not available to the customer, as they are in department stores. Based on the analysis of mall shop water use for those shops that had at least 100 working days of data, the mean water use was determined for each of 24 mall shop classifications as presented in Table A-74. As shown, a significant amount of variation was found within and between classifications. The authors further analyzed the data using linear regression techniques and found the following type of relationship most useful and predictive of water use,

where, $W = i + SA$

W = Water Use, L/day

i = Intercept in L/working day (4)

S = Coefficient expressed in L/working day/m^2

A = Gross store area, m^2

The values determined for i and S for those classifications with sample sizes of four or more are shown in Table A-75. The expression and values determined appear valid based on the high correlation coefficients.

TABLE A-74. SHOPPING CENTER COMPONENT STORE WATER USE (McCuen, Sutherland and Kim, 1975)

Type of Store	No. of Store	Mean (L/day)	S.D. (L/d)	Range of Store Area (m^2)	Mean (L/day/m^2)	S.D. (L/day/m^2)
Healthfood	1	135	-	114	11.9	-
Mens Clothing	20	483	499	86-1670	9.1	3.2
Womens Clothing	32	288	204	74-985	10.5	5.9
Hosiery	1	584	-	114	51.3	-
Shoes	28	256	138	588-5860	11.1	6.5
Wigs	2	246	59	1020-2560	34.5	3.7
Uniforms	2	399	359	70-141	33.4	17.8
Carpets	2	144	15.8	105-281	9.8	7.2
Cutlery	1	102	-	98	10.4	-
Appliances	4	618	808	187-2480	7.2	0.61
Music	4	415	231	1020-3720	15.4	2.5
Sporting Goods	1	218	-	496	4.4	-
Books	3	192	36.0	319-602	4.8	1.8
Jewelry	7	340	144	1130-3530	20.5	10.6
Toys	3	233	104	106-490	8.5	2.3
Cameras	2	196	82	67-131	19.9	1.0
Gifts	12	182	75	539-2400	8.8	3.6
Fabric	7	443	181	1900-6510	7.2	1.5
Art Supplies	2	147	63	134-293	7.1	0.8
Cosmetics	1	129	-	48	26.7	-
Art Gallery	1	108	-	351	3.1	-
Bath Goods	1	241	-	355	6.8	-
Gourmet Food	1	557	-	344	16.2	-
Opticians	2	154	71	71-122	15.6	1.5

TABLE A-75. SHOPPING CENTER COMPONENT STORE WATER USE RELATIONSHIP PARAMETERS (McCuen, Sutherland and Kim, 1975)

Type of Store	No. of Stores	i (L/working day)	S (L/working day/m^2)	Correlation Coefficient
Mens Clothing	20	-19.0	0.927	0.798
Womens Clothing	32	124.9	0.451	0.634
Shoes	28	92.9	0.610	0.707
Appliances	4	-14.6	0.740	0.999
Music	4	-26.3	1.672	0.967
Jewelry	7	166.1	0.879	0.777
Gifts	12	80.6	0.460	0.477
Fabrics	7	83.6	0.569	0.934

To facilitate comparison of the information gathered concerning shopping center flows, the average results of each study have been listed in Table A-76.

TABLE A-76. SHOPPING CENTER FLOW COMPARISON

Study	L/day/m^2
Anonymous (1961)	8.13
Searcy and Furman (1961)	8.13
Hubbell (1962)	7.73
Linaweaver, Geyer and Wolff (1967)	6.67
Feherty (1974)	7.55
McCuen, Sutherland and Kim (1975)	1.83 - 2.93

As shown in Table A-76, there is surprisingly good agreement between the average results determined in the various studies. The flow estimates presented in the general guideline tables are somewhat lower, typically equal to 4.07 L/day/m^2 (0.10 gal/day/ft^2).

Sports Facilities--A study investigating the wastewater flow generated by persons attending sports events was conducted by Pearson and Nesbitt (1975) at the Pennsylvania State University football stadium. Between 1968 and 1973, flow records were obtained at 15-minute intervals for 19 days on which football games were played. Game attendance on these days ranged from 38,600 to 59,980 with a mean of 51,340. The following flows were determined:

Average Quantity = 4.5 L/spectator
Standard Deviation = 6.4 L/spectator
Mean Peak Flow = 34 L/spectator/day (1 hour)
= 60 L/spectator/day (15 min.)
Peak Flow Standard Deviation = 7.6 L/spectator/day (15 min.)

The guideline tables include estimated flows from spectators at sports events and festivals typically equal to 3.8 to 19 L/spectator/day.

Theaters--Information on the water use and wastewater production at movie theaters (both auditorium and drive-in types) has been produced by the Oakland County Department of Public Works, Michigan (Ringler, 1975). Water use was monitored at both types of theaters for two years, producing the results shown in Table A-77. The average flow per seat from an auditorium type theater with air conditioning was found to be 2.7 gallons and the average flow per car space from a drive-in theater was found to be 5.0 gallons (both are based on an assumed 8-hour operating day).

Suggested design flows for theaters are typically 11.3 to 18.9 L/day (3 to 5 gal/day) per seat or car space.

TABLE A-77. THEATER WATER CONSUMPTION (Ringler, 1975)

	Type of Theater	
Parameter	Auditorium (L/seat/hr of oper.)	Drive-in (L/car space/hr of oper.)
Number of Theaters	13	4
Average Flow	1.28	2.34
Standard Deviation	1.17	1.74
Range	0.33 - 3.90	0.71 - 4.12

Discussion--

Predicting wastewater loadings for commercial establishments and public service facilities is a very complex task due to several factors;

1. There are a relatively large number of diverse categories, for which wastewater characterization data may be desired.

2. The inclusion of potentially diverse establishments within the same category (such as bars) produces a potential for large variations in waste generating sources, which could yield highly variable waste loadings. To avoid this problem, subcategories and classes of a general category should be utilized to aid predictability. However, this increases the number of groups of establishments to be considered. Further, many intangible influences such as location, popularity and price may result in wastewater variations between two otherwise similar establishments.

3. There is considerable difficulty in presenting characterization data in a readily applicable, but predictively accurate manner. The most readily applicable basis for predicting wastewater loadings, such as an easily identifiable physical parameter, may unfortunately not be predictively accurate.

In order to assess the sufficiency of the existing characterization data and to aid in determining the type of additional studies needed, a basis for assessment is required. In the context of this study it was assumed that to be considered "sufficient" the characterization data should provide an estimate of design loadings for small wastewater treatment and disposal facilities for the types of establishments under study. Although a wide variety of pollutant parameters are of interest, the following priority list of those parameters most often utilized in facilities design, was judged to be appropriate:

Priority	Parameters
1	Average Daily Flow
2	Daily Flow Variation (Peak Daily Flow)
3	Hourly Flow Variation
4	Average BOD_5 or COD
5	Average Suspended Solids
6	Average Nitrogen and Phosphorus
7	Wastewater Quality Variations

In terms of these seven parameter groupings, sufficiency can be viewed as a matter of degree, varying from no data available to information available for all the listed parameters, for each establishment under study.

In addition to simply having data available for the various parameters and establishment categories, it is important that the data were generated and presented in a predictively accurate and useful manner. Even if a given set of data is complete and accurate for the establishments at which it was generated, if the data cannot be employed to yield a reasonably accurate estimate of wastewater loadings at other, categorically similar establishments, the data is of marginal utility as a predictive tool.

A review of the existing characterization information readily indicates the type and extent of the data available for the establishment categories under study. The general guideline tables present lengthy lists of establishments which are often divided into sub-categories to avoid the diversity of establishments possible within descriptively, broad classifications such as schools, restaurants and theaters. In these tables design estimates are given for daily waste flow volume, and occasionally duration of flow, BOD_5 and suspended solids. The data are typically expressed as a function of some form of patronage or a physical characteristic of the establishment.

Unfortunately, the primary source of much of the guideline data is generally obscure or even unknown altogether. In many cases, a given guideline table has been reproduced, possibly with minor modifications, from one source to another, to another, etcetera. In each transfer, any documentation which may have accompanied the original data is diminished. Even when the original source of data was identified, attempts to determine the data generation methods were usually futile due to a lack of initial documentation or loss of documentation with time. Thus, any assessment of the reliability of this guideline information for predictive use is difficult, necessarily based on its "reasonableness" and individual statements concerning its "suitability." On the one hand, many regulatory individuals claimed the guideline tables they were employing had proven very satisfactory while on the other hand, accompanying many of the guideline tables were statements encouraging the use of actual monitoring of a similar establishment in preference to the guideline data.

As noted, general guideline information has generally been expressed as a function of some parameter related to patronage or a physical characteristic of the establishment. Examples of each include per meal served for restaurants and per alley for bowling alleys. At first glance, it would seem that a parameter related to patronage would be more accurate from a predictive standpoint than a physical characteristic. However, the application of data expressed as a function of patronage is often difficult since an estimate of patronage is required.

The individual establishment field studies have provided a variety of data for the categories under study through methods varying from the review of water utility records to in situ wastewater monitoring. Certain establishments have received considerable attention while others have received little. The availability of data for selected parameters and establishment categories varies considerably depending upon the parameter and establishment of interest. In general, even without consideration for methods or data presentation, there is a lack of data for many establishment categories. Further, the reliability of the available data is often questionable due to the manner in which the data was generated. For example, one or two daily composite samples of the wastewaters produced by a single establishment is of questionable predictive accuracy. At best, this type of data should serve only as a rough guideline.

As indicated by the summary of the existing information and the preceding discussion, a varying amount of characterization data is available for the commercial establishments and public service facilities selected for study. Depending on the parameter and establishment category of interest, the extent of the data base varies considerably. To a greater or lesser degree, however, it appears that further characterization studies are necessary for almost all of the establishment categories under study. Before proposing the form and extent of any studies, however, it was deemed necessary to review and consider the overall problem itself, namely, the characterization of wastewater loadings from commercial and public service establishments. More specifically, the methods used to generate and present characterization data had to be considered in the context of their providing useful estimates of the wastewater loadings from categorically similar establishments.

Analysis of Characterization Approaches

The generation and presentation of characterization data in a predictively useful and accurate manner is of prime concern. Even when a given set of data accurately represents the wastewater loadings for the establishment(s) at which it was generated, if the data is presented in such a way as to make it very difficult to apply to a categorically similar establishment, it is of marginal utility. With this in mind, this section is devoted to an evaluation of various approaches for generating and presenting characterization data.

At most any establishment or facility serving a transient population, there appear to be two major collective factors which contribute to the water use and waste production: (1) basic operation and employee personal activities and (2) direct or indirect contributions related to the patrons served. At any given establishment, a certain minimum of water-using and waste producing activities occur as a matter of basic operational routine. In addition to this contribution, and most likely the major of the two, is that waste contribution which results from the presence and activities of establishment patrons. Included are indirect as well as direct contributions. Indirect contributions stem from activities performed by the employees for the patron, such as food preparation, while direct contributions are the result of patron activities, such as restroom use. Further, due to variations in patronage throughout the day, week, and year, it would seem reasonable to expect concomitant variations in the water used and wastewater generated.

Of interest then, is a rational approach to take into account these factors and provide reasonable estimates of the expected wastewater loadings from the types of establishments under study. In an attempt to identify such a rational approach, an analysis of various approaches for generating and presenting characterization data for establishments serving a transient population was conducted.

Daily Quantitative and Qualitative Characteristics--

An accurate estimate of average and peak daily flow volumes would seem to be of highest utility in facilities design compared to the other parameters of interest. Also of interest are the average and peak daily contributions of selected chemical/physical parameters, such as BOD_5, suspended solids, nitrogen and phosphorus. In the following section, a detailed approach for estimating these parameter loadings will be presented first, followed by increasingly simpler approaches.

Approach 1--A variety of water using events within a given establishment commonly occur and result in the production of wastewater. A detailed approach to predicting wastewater loadings would involve delineating the characteristics of the wastewater produced by the component events which occur within the type of establishment under study. For each establishment category, the following type of prediction equation would result:

$$C = [N_{Oi}C_{Oi}] + [N_{Ej}C_{Ej}][E] + [N_{Pk}C_{Pk}][P] \tag{5}$$

Where, C = Total daily contribution of a selected parameter
N_{Oi} = Number of operational occurrences per day of event i
C_{Oi} = Contribution per occurrence of event i
N_{Ej} = Number of occurrences per day per employee of event j
C_{Ej} = Contribution per occurrence of event j
E = Number of employees per day
N_{Pk} = Number of occurrences per day per patron of event k
C_{Pk} = Contribution per occurrence of event k
P = Number of patrons per day

A major advantage of this approach is that it only takes into account the component events occurring at the establishment in question when identifying the expected wastewater loadings.

The development of this type of predictive equation for a given category of establishments would require several types of information obtained simultaneously at each of several establishments within the category. The characteristics of the wastewater produced per occurrence for each component event would have to be determined as well as the frequency of occurrence of each component event. Once developed, the predictive equation could be applied to a given facility within the category in question by making an estimate of the number of employees to be present and patrons served during a selected time period.

Data is available on the characteristics of certain wastewater events such as toilet flushing, bathing and clotheswashing, as a result of the household wastewater characterization study discussed previously. However, much information is lacking for the variety of waste generating events which could occur at the establishments under study. Determining the characteristics of the component events and their frequencies of occurrence would be very difficult, requiring an extensive effort for even a single category of establishments, as is evidenced by the effort expended in the characterization of household wastewaters.

This type of approach to predicting wastewater loadings was utilized for residential households. The component waste generating events were identified and characterized as to their quantity/quality per occurrence and the frequency of occurrence per resident was determined for each. Thus, with knowledge of the type of the component events to be in a planned home and an estimate of the number of residents, a prediction of the wastewater loading from the home can be made.

Approach 2--This simplification of approach 1 groups the contributions of the component events into three categories: (1) operational; (2) employee; and (3) patrons. This approach is based on the assumption that within a given category of establishments, such as bars, certain waste generating events occur typically and that the operational, employee and patron contributions are sufficiently constant to yield the following type of predictive equation:

$$C = O + K_1 [E] + K_2 [P] \tag{6}$$

Where, C = Total daily contribution of a selected parameter
O = Daily operational contribution
K_1 = Employee contribution
E = Number of employees per day
K_2 = Per patron contribution
P = Number of patrons per day

To ensure the utility of this type of predictive equation, for a single establishment type a narrow categorization of establishments would be required to avoid excessive diversity. The equation parameters for a given category could be identified by an extensive monitoring program at each of several establishments within the category. Wastewater flow information could be identified through monitoring the water use or, better yet, the wastewater produced, with qualitative information generated through composite sampling. Simultaneously, the employees present and patrons served would have to be recorded. Regression analysis could then be applied to the data to yield the equation parameters.

To apply this type of predictive equation to a proposed establishment within a particular category would require an estimate of the employees present and patrons to be served during a selected time period, e.g., a day.

Approach 3--A simplification of approach 2 would include the employee contribution into a base flow contribution and a second constant contribution per patron. Basically, characterization data would be expressed as a linear function of patronage in the form of

$$C = K_1 + K_2 [P] \tag{7}$$

Where, C = Total daily contribution of selected parameter
K_1 = Base contribution
K_2 = Contribution per patron
P = Number of patrons per day

Approach 4--A modification of approach 3 would express the operational and employee contributions collectively as a function of the full-time employees and some physical parameter indicative of establishment size and operational characteristics. The number of fixture units appears to be a readily identifiable parameter of this type. Thus, the predictive equation would be of the following form:

$$C = K_1[E][FU] + K_2[P] \tag{8}$$

Where, C = Total daily contribution of a selected parameter
K_1 = Operational and employee contribution divided by the total establishment fixture units
E = Number of employees per day
FU = Total establishment fixture units
K_2 = Contribution per patron
P = Number of patrons per day

The development of the equations outlined in approaches 3 and 4 would be similar to that described for approach 2. Basically, monitoring of several establishments within a given category would include recording water use and/or wastewater flow, composite sampling of the wastewater for qualitative characteristics, recording employees present and patrons served, and (particularly for approach 4) noting the fixture characteristics of the establishment. Regression analysis of the data generated would yield the parameters for the predictive equations and indicate their variability.

Once developed, this type of predictive equation could be applied to a proposed establishment if an estimate of the employees to be present and the patrons served were made. In the case of approach 4, variability in local plumbing code requirements as to the fixtures per unit capacity should be noted prior to application.

Unfortunately, this type of approach has not been utilized in any way for any of the establishments under study. As a result, there is no existing specific information or data of this type. However, approaches 3 and 4 were evaluated at ski areas in Southern Washington and Western Oregon during the 1967 ski season (Clark, 1969). Although ski areas are not specifically one of the establishments under study, they are very similar in that they serve a transient population in a remote area. The general category "ski area" was judged to include too diverse a group of establishments and it was, therefore, divided into classes and subclasses, to facilitate predicting water use and wastewater production. The water use was monitored at two day-lodge ski areas, three overnight-lodge areas, and one combined day-overnight lodge. In addition, ski area patronage was identified for each monitoring period. At two of the lodges the wastewater flow was also monitored and samples were taken.

Initially, water use relationships were derived from the data for the day lodge and overnight lodges in the form of approach 3:

$$Q = K_1 + K_2 [P] \tag{9}$$

Where, Q = Daily flow
K_1 = Base flow constant
K_2 = Per patron contribution of Q
P = Number of total visitors for day lodges and number of overnight guests for overnight lodges.

The combined day-overnight lodge equation included an extra term to allow differentiation for more than one type of patron yielding the following expression:

$$Q = K_1 + K_2[P_1] + K_3 [P_2] \tag{10}$$

Where, Q = Daily flow
K_1 = Base flow constant
K_2 = Per visitor contribution of Q
P_1 = Number of visitors per day
K_3 = Per overnight guest contribution of Q
P_2 = Number of overnight guests per day

The values determined for K_1, K_2, and K_3 for each of the lodges studied are presented in Table A-78.

TABLE A-78. WATER USE RELATIONSHIP PARAMETERS, L/day (Clark, 1969)

Lodge Type	K_1*	95% Confidence Limits	
		K_2*	K_3*
Day Lodge			
1	13340	8.13 - 11.72	-
2	4140	2.19 - 4.16	-
Overnight Lodges			
1	13120	155 - 272	-
2	7940	102 - 155	-
3	2950	41.6 - 93.2	-
Combined Lodge	25520	3.78 - 6.8	37.8 - 136

* For equation (9), $Q = K_1 + K_2[P]$, for day lodges and overnight lodges and equation (10), $Q = K_1 + K_2[P_1] + K_3[P_2]$, for combined lodges. All values are expressed in L/day.

Recognizing that the base flow, K_1, was a function of the size of the facility, Clark normalized K_1 by dividing by the number of full-time employees and fixture units. The resulting values of the normalized K_1 were quite consistent, ranging from 5.63 to 7.30 L/day/employee per fixture unit (FU) (1.49 to 1.93 gal/day/employee per fixture unit) with a mean of 6.54 L/day/employee/FU (1.73 gal/day/employee/FU). The form of the resulting predictive relationshop, essentially that of approach 4, was, $Q = K_1(E)(FU) + K_2(P_1) + K_3(P_2)$ with $K_1 = 6.54$ L/day/employee/FU (1.73 gal/day/employee/FU) and the values of K_2 and K_3 selected from Table A-78 depending on the lodge type.

In monitoring wastewater production at two lodges Clark found that approximately 70% of the water used was returned as wastewater and the peak daily flow was approximately 10 times the average daily flow. Based on sample analyses for a variety of parameters and the application of linear regression techniques, predictive equations in the general form of Approach 3 resulted:

$$C = K_1(E) + K_2(P_1) + K_3(P_2) \qquad (11)$$

Where, C = Total daily contribution of BOD_5, COD, SS, TKN, or TPO_4
K_1 = Base value constant (daily contribution per employee)
E = Number of employees
K_2 = Contribution of C per visitor per day
P_1 = Total visitors
K_3 = Contribution of C per overnight guest per day
P_2 = Total overnight guests

The values determined for the constants for BOD_5, TSS, TKN and TPO_4 are listed in Table A-79.

TABLE A-79. POLLUTANT CONTRIBUTION EQUATION PARAMETERS, L/cap/day (Clark, 1969)

	K_1*	K_2*		K_3*	
Pollutant		Mean	95% C.L.	Mean	95% C.L.
BOD_5	77.2	2.59	±3.54	18.6	± 59.9
COD	109	7.63	±6.04	38.3	±104
TSS	26.8	0.141	±5.74	133	± 95.3
TKN	- 21.5	2.81	±4.77	31.6	± 86.3
TPO_4	0.445	-0.073	±0.318	6.99	± 5.27

* For equation (11), $C = K_1[E] + K_2[P_1] + K_3[P_2]$. All values shown are in g/cap/day.

Several points concerning this study are worthy of note. The category of ski areas was believed to be too broad and was divided into narrower subcategories. For one of these subcategories, two types of patrons were identified and included in the predictive equation. The monitoring study conducted to develop predictive equations for this single category of establishments was very detailed. Water use and wastewater production flow monitoring, composite wastewater sampling, and a record of patrons served were accomplished at several establishments for a period of about six months. The variability of the equation parameters for flow is fairly low with reasonably narrow 95% confidence intervals (Table A-78). However, the parameters for the qualitative equations are highly variable with wide 95% confidence intervals (Table A-79). In order to apply the developed equations to a proposed facility, the designer or engineer must be able to make a reliable estimate of the employees and patrons expected.

Approach 5--A major simplification of the previous approaches is to express the wastewater loadings totally as a function of the number of patrons served. This approach is based on the assumption that all of the water used and wastewater produced is somehow indirectly or directly the result of the patrons served. Although the operational and employee contributions may exhibit significant short-term variations in their relationship to the number of patrons served, over the long-term and with regard to a design loading based on some form of maximum patronage, the variations in operational-employee contributions are most likely lower and in some cases insignificant. The form of this predictive equation would be:

$$C = K_1[P] \tag{12}$$

Where, C = Total daily contribution of a selected parameter
K_1 = Average contribution per patron served
P = The number of patrons served per day

The information necessary to quantify the parameter, K_1, includes water use and/or wastewater flow data, wastewater quality information, and a simultaneous record of the patrons served. The application of this type of equation, after determining the value for K_1, would require only an estimate of the number of patrons expected.

This type of approach is commonly used for residential dwellings. Although information is available to predict the wastewater loadings on a component event basis as discussed in approach 1, the contribution per resident is often set equal to that of the component events found in a "typical household." A prediction of the wastewater loading is made for a typical household by estimating the number of residents.

This approach has also been applied to several of the establishments under study in the field investigations previously discussed. For example, the water use/wastewater flow from restaurants is commonly expressed per meal served (i.e. per customer) and for schools, per student.

There are several major problems associated with the development and application of the five previously described predictive approaches. In regard to their development, for each category of establishments a number of such establishments would have to be monitored for a number of days to yield statistically significant results. The monitoring would necessarily include not only water use and/or wastewater production, but also employees present and the patrons served. Upon development, the application of a given predictive equation to a particular establishment would require an estimate of the patrons to be served and possibly the employees to be present over a selected time period. Obviously, if this estimate of patronage is not reasonably accurate, the value of detailed predictive equations is diminished greatly. This raises the question of how expected patronage can be accurately estimated.

Considering any establishment serving a transient population it seems likely that actual patronage varies considerably from day to day at a particular establishment and between establishments within the same category. This is most likely due to quantifiable factors such as services rendered and establishment size, as well as subtle, intangible influences, such as location and aesthetics. As a result, an estimate of actual patronage for a selected time period is very difficult to accomplish. However, most design is based on some form of maximum loading rather than a form of a variable, actual loading. Therefore, an estimate of maximum patronage of some form would be desired. For certain types of establishments, such as motels and theaters, an estimate of maximum patronage could be made due to a finite, identifiable capacity and a known patron turnover rate. However, for other establishments, such as bars and service stations, the capacity is not finite and identifiable, and the patron turnover is highly unpredictable. For these latter type of establishments, an estimate of even maximum patrons to be served could be very difficult to make.

One approach to identifying maximum patronage for a given establishment category would involve generating a patronage distribution. This approach could be applied and used not only for those establishments with a finite capacity and identifiable turnover, but also for those without such characteristics. If one were to monitor patronage versus time at a given establishment within a selected category, such as bars, the variations in patronage could be identified on an hourly and/or daily basis. If several categorically similar establishments were monitored over several days (from low to high patronage days) and the data were normalized by dividing by an establishment characteristic, such as seats, car spaces, square footage, etcetera, the data could be grouped together and plotted. Ideally, a relationship such as that shown in Figure A-33 would result. With the type of patronage information shown in this figure for an establishment category, the designer could select a design patronage, say 90%, thereby yielding the patrons/time/physical characteristic. With a knowledge of the specific physical characteristic at the establishment in question, the design patronage could then be identified. This design patronage could then be utilized with one of the previously described predictive equations to estimate wastewater loadings.

Approach 6--The difficulties associated with the generation of the previously discussed predictive equations, as well as their subsequent application, are some of the reasons that they have not been utilized to any significant extent. Although the characterization data in a few cases has been generated and expressed as a function of the patrons served, the typical unit of contribution has been a readily identifiable physical characteristic such as seats, car spaces, square footage, etcetera, depending on the establishment category in question. Expression of characterization data in this manner is much simpler and its application more readily accomplished.

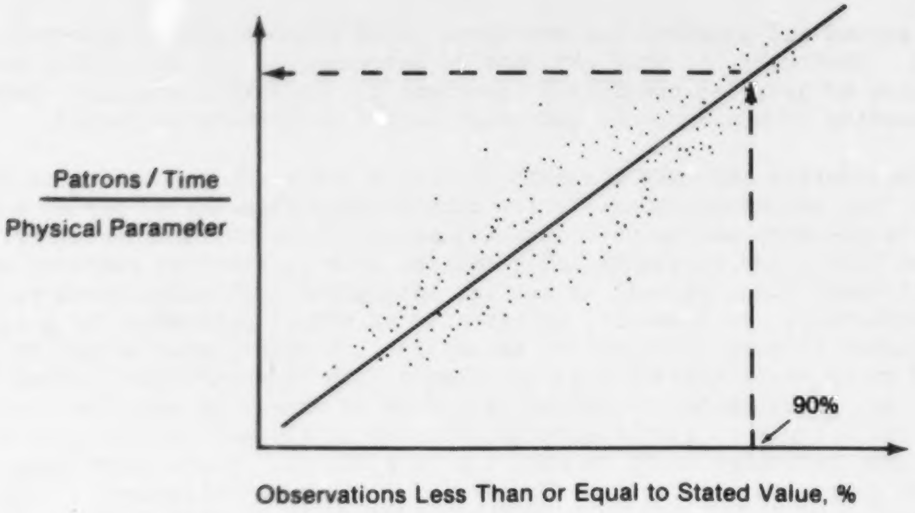


Figure A-33. Hypothetical patronage distribution for a given establishment category.

In using a physical characteristic to express wastewater characterization data, the major assumption is that a properly selected physical characteristic is indicative of potential patronage and water use/wastewater production. At first glance, this assumption does not appear to be as appropriate as an approach based on patronage. Reference is made to the field studies conducted on each establishment category which indicated that results expressed as a function of a physical characteristic exhibited a very large variation between categorically similar establishments (e.g., laundromats) while those expressed as a function of patronage in some form were fairly consistent (e.g., schools). Even very similar establishments can experience significantly different patronage due to intangible influences.

However, since designs are generally based on maximum loadings as a result of maximum patronage, it is possible that a certain physical characteristic may be indicative of the expected maximum patronage. This seems reasonable since some physical characteristics such as seats, car spaces, square footage, fixture units or a combination thereof, do establish a maximum patronage, at least for a given turnover time period. As noted, for certain establishments, this time period is readily identifiable, while for others it is very difficult to identify or even estimate. If the water use and/or wastewater production were monitored at a given establishment while it encountered a maximum patronage, it may in fact be appropriate to express the characterization data as a function of a physical parameter. These data would then represent a maximum loading, rather than an average loading.

The underlying assumption of this approach is that patronage is a function of a selected physical parameter. More accurately, the assumption is that the maximum patronage is a function of a selected physical parameter. Expressed in equation form, it would appear as:

$$P_d = K[PP] \tag{13}$$

Where, P_d = Maximum patronage
K = Patronage coefficient
PP = Physical parameter (number of seats, square footage, etcetera) which provides best correlation

For certain establishment categories which have a definite capacity and identifiable turnover such as schools, motels and theaters, the value of K can be estimated fairly well. For others such as bars, bowling alleys and service stations, an estimate of K would be very difficult due to a nebulous capacity and turnover. For these latter establishment types, the development of a patronage distribution curve as previously discussed may be necessary to accurately estimate K. Assuming values for K were identified for a particular establishment category, the relationship for P_d could be used to estimate maximum patronage for a proposed establishment. This value could then be used in the previously described predictive equations, to yield an estimate of maximum wastewater loadings.

Approach 7--A gross simplification of the previously described approaches and one which has commonly been used, is to ignore patronage and generate wastewater characterization data based on a selected physical parameter. An equation such as the following results:

$$C = K[PP] \tag{14}$$

Where, C = Total daily contribution of selected parameter
K = Parameter expressed as a function of PP
PP = Selected physical parameter

If the value of K were determined at one or more establishments within a given category which were experiencing a high level of patronage, it would be useful for design purposes. However, if K were determined at establishments encountering a very low patronage, it would be incorrect to use this K for design.

To generate this type of relationship, it would be necessary to monitor water use and/or wastewater flow on a selected time basis (e.g., daily) for volume information and to composite samples from the wastewater for qualitative data. If a number of establishments were monitored over a number of days, and the data were expressed as a function of a common physical parameter, the data could be combined to yield statistical results for the category in question. In regard to applying these results in a predictive manner, it would be misleading and incorrect unless the data were generated under high patronage conditions. To ensure that the data were generated under high, design-type use, one could either specifically select monitoring days where high use would be encountered or monitor over a number of days so that high use days would be included. Then, by having a knowledge of the selected physical characteristic at a proposed establishment, the value for K of the establishment category in question could be used to yield an estimate of the maximum wastewater loadings expected.

As noted previously, this type of approach has commonly been used for many of the establishments under study, as well as for others. The obvious reason for this is the comparative simplicity in data generation and application. Unfortunately, the use of this approach without regard to the basis for the approach has often led to high variability in the data for a given establishment category.

Approach 8--A final approach includes a minor alteration of approach 7 in that a base flow constant would be included to more accurately represent the data. An equation similar to that of approach 3 results:

$$C = K_1 + K_2[PP] \tag{15}$$

Where, C = Daily contribution of Q, BOD_5, TSS, . . . per time
K_1 = Base flow constant
K_2 = Daily contribution per physical parameter
PP = Selected physical parameter

The generation and application of this type of approach is similar to that of approach 7.

McCuen, Sutherland and Kim (1975) found this suitable for predicting shopping center water use. The values of parameters equivalent to K_1 and K_2 were determined for a number of shopping centers (department stores and mall shops) utilizing water use information and physical characteristics. For a complete discussion of this example, reference is made to the individual establishment field studies section on Shopping centers.

Daily Qualitative Characteristics--

Within a reasonably narrow establishment category, establishments should typically contain similar waste generating events, although perhaps different in number. The occurrence of these waste generating events as a result of basic operation and patron related activities results in the production of wastewater. At a given establishment the concentration of various pollutants in the wastewater stream would be expected to vary during the day due to the intermittent occurrence of different events. Similar variations may also exist from day to day, especially between high and low patronage days due to the influence of operational flows and clean water flows from automatic events such as urinal flushing. Further, daily variations may result due to special events such as a potluck dinner at a church. Between categorically similar establishments, daily effluent concentrations determined by short-term monitoring could differ for the above reasons as well as some differences in waste generating events.

Although variations may occur in the strength of a wastewater at a given establishment and between categorically similar establishments, it would seem that for the purpose of estimating the chemical/physical characteristics of an establishment category's wastewater, representative daily composite concentrations should suffice. These concentrations could be determined through some form of composite wastewater sampling at several establishments. These chemical/physical concentrations could be used in combination with the daily waste flow volume estimated by one of the detailed approaches described earlier to yield an estimate of pollutant mass/day if desired.

A comparison of this concentration approach versus a detailed mass approach may be made utilizing the data generated by the wastewater characterization study at winter ski areas (Clark, 1969). As part of the study, the wastewater flow volume was measured and composite samples taken at each of two overnight-day use ski areas over approximately two weeks. Analyses were performed for BOD_5, suspended solids and other standard chemical/physical parameters. A summary of the values determined for selected parameters is shown in Table A-80. As shown, the day-to-day variations in the concentration of any of the parameters is not excessive, nor is the variation between the two establishments.

TABLE A-80. SKI LODGE WASTEWATER CHARACTERISTICS
(Clark, 1969)

	Ski Lodge A		Ski Lodge B		
Parameter	Mean	Std. Deviation	Mean	Std. Deviation	Average
Samples	9-12	-	15-16	-	-
BOD_5, mg/L	395	126	382	170	388
TSS, mg/L	321	177	372	208	346
TKN, mg/L	76.6	-	80	29.3	78
TPO_4, mg/L	12.7	2.4	13.2	6.6	13
Flow, L/day	31,600 L/day	-	56,400 L/day	-	-

Using the individual wastewater characterization data and records of daily visitors and overnight guests which had been obtained simultaneously in combination with the employee and fixture unit characteristics of each establishment, predictive equations of the following form were generated:

$$\text{Flow} = a(E)(FU) + b(TV) + c(G) \qquad (16)$$

$$\text{Pollutant Mass} = x(E) + y(TV) + z(G) \qquad (17)$$

Where, a,b,c,x,y,z = Equation parameters
E = Employees
FU = Establishment fixture units
TV = Total daily visitors
G = Total overnight guests

The predictive equations suggested for use in design for flow, BOD_5, and TSS were as follows:

$$\text{L/day} = 6.54\,(E)(FU) + 6.80(TV) + 136(G) \qquad (18)$$

$$\text{g } BOD_5\text{/day} = 77.2(E) + 6.13(TV) + 78.5(G) \qquad (19)$$

$$\text{g TSS/day} = 26.8(E) + 5.90(TV) + 132(G) \qquad (20)$$

An example will serve to compare the use of an average pollutant concentration and a detailed flow equation compared to a detailed mass equation to estimate daily pollutant contributions from a given establishment.

Lodge A:

Employees = 15
Fixture Units = 300
Daily Visitors = 2225
Daily Overnight Guests = 75

Detailed Pollutant Mass Equation:

$$\begin{aligned} \text{g BOD}_5\text{/day} &= 77.2(15) + 6.13(2225) + 78.5(75) \\ &= 20{,}700 \text{ g BOD}_5\text{/day} \end{aligned} \tag{19}$$

$$\begin{aligned} \text{g TSS/day} &= 26.8(15) + 5.90(2225) + 132(75) \\ &= 23{,}430 \text{ g TSS/day} \end{aligned} \tag{20}$$

Average Pollutant Concentration & Detailed Flow Equation:

$$\begin{aligned} \text{L/day} &= 6.54(15)(300) + 6.8(2225) + 136(75) \\ &= 54{,}770 \text{ L/day} \end{aligned} \tag{18}$$

$$\text{Average BOD}_5\text{/day} = 388 \text{ mg/L}$$

$$\begin{aligned} \text{g BOD}_5\text{/day} &= \frac{(388 \text{ mg/L})(54{,}770 \text{ L/day})(8.34 \text{ lb/gal})(454 \text{ g/lb})}{(3.78 \text{ L/gal})(10^6 \text{ gal/MG})} \\ &= 21{,}290 \text{ g BOD}_5\text{/day} \end{aligned}$$

$$\text{Average TSS} = 346 \text{ mg/L}$$

$$\begin{aligned} \text{g TSS/day} &= \frac{(346 \text{ mg/L})(54{,}770 \text{ L/day})(8.34 \text{ lb/gal})(454 \text{ g/lb})}{(3.78 \text{ L/gal})(10^6 \text{ gal/MG})} \\ &= 18{,}980 \text{ g TSS/day} \end{aligned}$$

As indicated by this example, the use of average concentrations determined through daily composite sampling in combination with a reasonable estimate of flow volume yielded a mass/day contribution very similar to that provided through use of a very detailed mass/day prediction equation.

A further ramification of this concentration-volume approach is that it may be possible to predict the strength of the wastewater produced by different establishment categories if the waste generating sources and patron activities are reasonably similar. What may result are larger establishment groupings, including several establishment categories, for the purpose of estimating wastewater strength. For example, it is conceivable that the strength of the wastewater produced by bars, bowling alleys, and theaters may be sufficiently similar to form one such group. Unfortunately, data are lacking to support or refute this possibility.

This type of approach is commonly utilized for municipal wastewater facilities design, with average concentrations and their ranges utilized for parameters such as BOD_5 and suspended solids. To convert these concentrations to a mass, an estimate of waste flow volume is made and a simple conversion is applied. Certain of the qualitative information presented in the general guideline tables in terms of mass/day/parameter would appear to have been generated in this manner, i.e., typical domestic wastewater concentrations have been converted to mass values by utilizing an estimated flow volume.

Peak Flow Characteristics--

The intermittent occurrence of various waste generating events within an establishment or facility results in the production of a wastewater which varies in both volume and strength with time. The approaches discussed in the previous sections have dealt primarily with the prediction of waste loadings on a daily basis rather than on a short-term basis, such as per hour. For the various characterization parameters, short-term peak loadings appear to be of marginal utility with the exception of flow volume. Peak sewage flows are often critical to design, and methods for their estimation merit further discussion.

The short-term peak sewage flow from a given establishment is a function of the types and numbers of waste generating fixtures present, such as showers, sinks, toilets, dishwashers, washing machines, etcetera. Obviously, the maximum peak loading which could physically occur would result under simultaneous discharge from all fixtures present. This peak rate could be easily estimated with a knowledge of the fixtures present and their typical flow rates and/or volume per use. However, the likelihood of all fixtures discharging at once would seem to be very small. Rather than this maximum peak flow, it would seem that an "expected peak flow" would be more useful and appropriate.

To identify expected short-term peak flows, there appear to be two basic approaches. The first is a probabilistic approach based on the types and numbers of fixtures present, typical flow rates and volume per use, and the probability of their simultaneous occurrence (in part or in total). The second approach is more of a brute-force monitoring approach yielding expected peak flows for a given establishment type as a result of water use/waste flow monitoring versus time of day. A discussion of each approach follows.

Probabilistic approach--This type of approach is basically that developed by Hunter (1940, 1941) commonly referred to as the "Fixture-unit Method." Originally developed for predicting peak water demands within buildings, it has commonly been used to also estimate peak sewage flows. A discussion of this approach was given in the section summarizing the existing information. Briefly, the water demand through a fixture was seen to be a function of the flow rate, length of use and frequency of use. The fixture unit was defined as a water demand equivalent to 7.5 gallons per minute and various fixtures were then assigned a certain number of fixture units based upon their water use characteristics. Utilizing probability theory, Hunter developed curves relating the expected peak water demand to the number of fixture units present (refer to Table A-46 and Figure A-15).

Field monitoring--Continuous water use/waste flow monitoring at several establishments within a given category over a sufficient length of time would yield information on peak flow (e.g. peak hourly flow). Further, periods of low or no flow could be identified. The major concern with this approach is that the establishments monitored be representative of those typically included in the given establishment category and that the monitoring be conducted over a sufficient length of time so that the peak flows are actually encountered.

As part of several of the establishment field studies previously discussed, peak flows have been identified and occasionally flow hydrographs developed. For the most part, this monitoring has been restricted to water usage, due to the relative simplicity of gathering this type of data. Although water usage does not necessarily equal wastewater production in either volume or flow pattern, peak water usage is often indicative of peak sewage flow.

To allow a general review of selected peak flow data determined in the establishment studies, Table A-81 has been prepared. The ratio of peak hourly flow to mean daily flow determined through in situ monitoring has been calculated where possible and presented. As shown, for certain establishments, peak hour ratios have not been identified, while for others more than one study has been conducted. It is interesting to note that where more than one study has been conducted on a given establishment type, the peak hour/mean daily values determined are reasonably consistent. However, between very different establishments such as restaurants and churches, there is a significant difference in the peak flow ratios determined.

TABLE A-81. PEAK HOURLY FLOW RATIO DATA

Establishment	Ratio Peak Hour/Mean Daily
Bars/Taverns	-
Bowling Alleys	-
Campgrounds & Picnic Parks	-
Churches	24.0
Country (Golf) Clubs	-
Laundromats	-
Marinas	-
Motels	4.5, 6.9
Restaurants	2.0, 2.7
Schools	3.2-5, 3.3-3.7, 9.1-18.2
Service Stations	19.5
Shopping Centers	2.6, 2.0, 2.5
Sports Facilities	7.5
Theaters	-

Although it is difficult to draw conclusions from the data presented, it appears that those establishments with relatively high numbers of fixture units and reasonably steady daily patronage such as schools, shopping centers, and restaurants encounter a lower peak flow ratio than those with relatively low numbers of fixture units and variable daily patronage, such as service stations and churches. This seems reasonable, since the peak hourly flow is not affected by low daily-use days, whereas the mean daily flow may be significantly reduced.

Discussion--

A summary of the approaches previously discussed is presented in Table A-82. These were reviewed and compared in an attempt to identify which should in general be used to estimate the loading of a given parameter. Consideration was given to (1) the type and degree of monitoring required for development, (2) the information necessary to apply an approach once developed, (3) the accuracy of prediction expected, and (4) previous successful applications for establishments such as those under study. Based on this analysis, certain types of approaches have been identified as most appropriate and are recommended as described in the following sections.

Daily quantitative characteristics--Consideration of the existing characterization data and the preceding analyses of various approaches to generate data in a predictively useful manner clearly indicated that there are two, distinct levels of predictive accuracy possible depending on the type of approach employed. With regard to estimating average and maximum daily flow volumes, the lower level of predictive accuracy appears to result from the use of approaches which generate characterization data as a function of an establishment physical parameter such as square footage, seats, car stalls, etcetera. An exception to this occurs when an estimate is desired for maximum daily flow and the establishment category in question possesses a finite capacity and known turnover rate. Since the maximum patronage is a function of an establishment size characteristic, the maximum daily flow, produced under maximum patronage, is likewise a function of the establishment size. The higher level of predictive accuracy appears to result when an approach is utilized which generates characterization data as a function of patronage.

The approaches presented and discussed for predicting average and peak daily waste flow volumes were divided into three groups of approaches, consisting of those based on: (1) component event wastewaters (A1), (2) patronage (A2 to A5), and (3) establishment size (A7, A8). All of the approaches within the same group require the same general monitoring efforts for development and/or information for application. Of these three groups, the first two appear to offer higher predictive accuracy but require greater developmental and application efforts, than the third. After consideration of all three groups, the second group (A2 to A5) which generates the characterization data as a function of patronage is felt to offer the proper balance between accuracy of prediction and developmental/application requirements and is, therefore, recommended.

Monitoring requirements to enable the development of the recommended approaches include simultaneous water use and/or wastewater flow volumes and patronage measurements. Additionally, establishment size would be noted as indicated by fixture units, seats, etc. Analysis of the data would then indicate which of the approaches, A2 to A5, provided the greatest accuracy of estimate. It should be noted that the monitoring would also allow analysis of the data for approaches A7 and A8.

TABLE A-82. SUMMARY OF CHARACTERIZATION APPROACHES ANALYZED

No.	Parameter Grouping/Approach	Eqn. No.	Monitoring Required During Development					Estimates Required for Application
			Waste Flow	Event Freq.	Patrons	Employees	Physical Parameter	
	Daily Quantitative & Qualitative Characteristics							
A1	$C=[N_{Oi}C_{Oi}]+[N_{Ej}C_{Ej}][E]+[N_{Pk}C_{Pk}][P]$	(5)	X	X	X	X	X	Events Patrons Employees
A2	$C = O + K_1E + K_2P$	(6)	X		X	X		Patrons Employees
A3	$C = K_1 + K_2P$	(7)	X		X		X	Patrons
A4	$C = K_1[E][FU] + K_2P$	(8)	X		X	X	X	Patrons Employees Physical Parameter
A5	$C = K_1P$	(12)	X		X			Patrons
A6	$P_d = K\ (PP)$	(13)			X		X	Physical Parameter
A7	$C = K\ (PP)$	(14)	X				X	Physical Parameter
A8	$C = K_1 + K_2\ (PP)$	(15)	X				X	Physical Parameter
	Daily Qualitative Characteristics							
A9	Use of Approaches under A	-						See 1-8 Above
A10	Average Concentrations	-	X					None
A11	Mass Conversion of Ave. Conc. Using Flow Estimate (1-8)	-						None
	Peak Flow Characteristics							
A12	Probabilistic Approach	-	X				X	Physical Parameter
A13	Field Monitoring (1-8)	-						See 1-8 Above

To apply any of the approaches A2 to A5 requires an estimate of expected patronage with approach A2 also requiring an estimate of employees and establishment size. As discussed previously, expected patronage, especially maximum patronage, can be estimated readily for certain categories of establishments such as motels and theaters due to a finite capacity and known patron turnover rate. However, for other establishments, such as bars and service stations, the capacity is not finite and the patron turnover is highly unpredictable, thus making patronage estimates very difficult to accomplish. For these establishments an empirical relationship of patronage and establishment size [P = f (size)] would have to be derived during the monitoring phase. With the patronage estimates, average flow could then be estimated using average patronage with peak daily flow the result of maximum daily patronage.

Daily qualitative characteristics--As indicated in the preceding analysis of characterization approaches, it appears appropriate to generate qualitative characterization data for a given category of establishments in concentration form. The generation of this data for a given category of establishments would require the collection and analysis of several flow composited samples from each of several establishments within the category. As noted previously, the category should be descriptively narrow and contain establishments possessing similar waste generating sources. Conversion of the concentrations to mass contributions could be accomplished by utilizing an estimate of the waste flow volume obtained as previously recommended. To apply the resulting characterization data would only require assigning the establishment in question to a particular establishment category.

Peak quantitative characteristics--The characterization of peak wastewater flow is recommended by continuous water use or wastewater flow monitoring during expected hours of peak flow. Expression of the peak flow appears most useful as a ratio of the mean daily flow measured. Thus, this monitoring should ideally be accomplished along with the monitoring conducted to characterize daily wastewater flow volumes. It is possible that this type of characterization approach may facilitate the development of a probabilistic approach similar to that developed by Hunter (1940, 1941), as discussed previously.

To summarize briefly, the following types of approaches are recommended for generating and presenting characterization data for the types of establishments under study in a predictively useful and accurate manner:

Daily Quantitative Contributions--

$Q_{ave} = f\ (\text{Patronage}_{ave})$

$Q_{max} = f\ (\text{Patronage}_{max})$

Patronage values required for application estimated based on establishment size or a derived relationship.

Daily Qualitative Characteristics--

The results of statistical analysis of flow-composited samples with mass contributions calculated.

Peak Flow Characteristics--

Continuous water use and/or wastewater flow monitoring with expression as peak to mean daily.

Characterization Data Generation

In the discussion at the close of the summary of the existing information section, the characterization data provided by previous field studies was briefly reviewed and considered as to its extent, but this was done without regard to the approach used to generate and present the data. However, the approach used to generate and present characterization data is of prime importance, and to a large degree, determines the predictive accuracy and reliability of the data. Thus, consideration of the existing data must necessarily include consideration of the approach utilized to generate the data. A review of the existing information from previous field studies, considering the amount of data available as well as the approach used for its generation, indicated the characterization data which was altogether lacking or in need of verification for each of the establishments under study and several key parameters. This information, as presented in Table A-83 shows a substantial amount of characterization data lacking or in need of verification.

At the onset of this study, it was anticipated that after compiling a comprehensive summary of the existing characterization data for the establishments selected for study, consideration of the summary would indicate the data which was lacking or in need of verification, and then field studies could be readily accomplished to provide the necessary data. However, after the existing characterization data was compiled and evaluated and an analysis of characterization methods was conducted, it became clear that proper characterization of the wastewaters from the study establishments was a considerably more difficult task than had been originally envisioned. Thus, it became obvious that this study could not hope to provide all the missing pieces of data due to finite resources and time constraints, but rather would indicate which pieces of data were lacking, recommend the proper approaches to generating the necessary data, and provide as much additional data as possible.

Data Generation Methods--

To provide additional characterization information, data was collected regarding quantitative and qualitative characteristics for several categories of establishments. To facilitate data gathering, the cooperation of two, large liquid waste pumpers/haulers was secured. It was discovered that many of the establishments under study employed holding tanks for disposal of their wastewaters and it was felt that this would facilitate monitoring efforts, both for quantitative, as well as qualitative characterization.

Quantitative Characterization--Attempts were made to include all of the establishments under study for which additional waste flow volume data was necessary in this effort. In total, 18 establishments within 7 of the categories under study were included. As a result of certain political considerations, measurements had to be obtained from holding tank pumpage records, which in most cases covered a two-year period. A list of the establishments included and their characteristics appear in Table A-84. Attempts were also made to secure the cooperation of several local establishments to allow the

TABLE A-83. CHARACTERIZATION DATA ALTOGETHER LACKING OR REQUIRING VERIFICATION*

Establishment Category	Quantitative			Qualitative			
	Mean Daily Flow	Max. Day / Mean Day	Peak Hour / Mean Day	BOD_5 or COD	Suspended Solids	Nitrogen	Phosphorus
Bars/Taverns	X	X	X	X	X	X	X
Bowling Alleys	X	X	X	X	X	X	X
Campgrounds		X	X		X	X	X
Picnic Parks		X	X		X	X	X
Churches				X	X	X	X
Country (Golf) Clubs	X	X	X	X	X	X	X
Laundromats							
Motels				X	X	X	X
Restaurants						X	
Drive-In Restaurants	X	X	X	X	X	X	X
Schools				X	X	X	X
Service Stations	X	X	X	X	X	X	X
Shopping Centers				X	X	X	X
Sports Facilities				X	X	X	X
Theaters	X	X	X	X	X	X	X
Drive-In Theaters	X	X	X	X	X	X	X

* Consideration given to amount of data available as well as the methods used for its generation and presentation. Data presented as general guidelines (e.g. guideline tables) have been excluded from this analysis.

monitoring of water use. However, a complete unwillingness to participate in any form by several establishments quickly discouraged these efforts.

TABLE A-84. ESTABLISHMENT CHARACTERISTICS - QUANTITATIVE CHARACTERIZATION

Establishment			
No.	Type	Characteristics	Period of Data
1	Bar/Tavern	50 seats, restrooms	1/74 - 12/74
2	Bar/Tavern	110 seats, restrooms	5/74 - 12/74
3	Campground	230 sites, comfort stations, laundry	5/75 - 9/76
4	Church	125 members, restrooms, kitchen facilities	1/74 - 9/76
5	Church	160 attendants on Sunday, restrooms, kitchen	10/74 - 10/76
6	Church	200 attendants on Thursday and Sunday, restrooms	4/74 - 9/76
7	Church	360 members, restrooms, kitchen facilities	9/74 - 8/76
8	Country Club	Golfing, 75 seat restaurnat, locker rooms/showers	11/74 - 10/76
9	Country Club	Golfing, 60 seat restaurant, locker rooms/showers	7/75 - 9/76
10	Country Club	Golfing, 700 seat restaurant, locker rooms/showers	10/74 - 10/76
11	Drive-In Restaurant	20 car stalls, 20 seats, restrooms	10/74 - 10/76
12	Restaurant	300 seats	10/74 - 10/76
13	School	160 students, restrooms	10/74 - 10/76
14	Gas Station	2 bays, 2 pump isles, restrooms	12/74 - 10/76
15	Gas Station	3 bays, 2 pump isles, restrooms	10/74 - 10/76
16	Gas Station	2 bays, 2 pump isles, restrooms	12/74 - 10/76
17	Gas Station	2 bays, 2 pump isles, restrooms	12/74 - 10/76
18	Gas Station	2 bays, 2 pump isles, restrooms	10/74 - 11/76

Qualitative characterization--Qualitative characterization was conducted at 14 establishments within five of the categories under study: churches, country clubs, restaurants, schools and service stations. A list of the establishments included and their characteristics is shown in Table A-85.

This characterization was accomplished through sampling of the wastewater in the holding tank serving each of the establishments. Each time the holding tank was pumped, a sample of the wastewater was collected. The sample was obtained in three portions, at the beginning, middle and end of pumping, off of a drain valve on the centrifugal pump located on the pumper tank. After collection, the samples were refrigerated until they were transported back to the University of Wisconsin for analysis. Each sample collected was analyzed routinely for chemical oxygen demand, total and suspended solids, total Kjeldahl nitrogen and total phosphorus according to procedures outlined in Standard Methods (1971). The results of the analyses were subsequently

TABLE A-85. ESTABLISHMENT CHARACTERISTICS - QUALITATIVE WASTEWATER CHARACTERIZATION

Establishment		
No.	Type	Characteristics
1(8)*	Country (Golf) Club	Golfing, 75 seat restaurant, locker rooms w/showers
2(10)	Country (Golf) Club	Golfing, 700 seat restaurant, locker rooms w/showers
3(4)	Church	125 members, restrooms, kitchen facilities
4	Church	-
5(12)	Restaurant	300 seats, restrooms
6	Restaurant	-
7(11)	Drive-In Restaurant	20 car stalls, 20 seats, restrooms
8	School	Elementary
9(17)	Service Station	2 bays, 2 pump islands, restrooms
10(15)	Service Station	3 bays, 2 pump islands, restrooms
11(18)	Service Station	2 bays, 2 pump islands, restrooms
12	Service Station	-
13(16)	Service Station	2 bays, 2 pump islands, restrooms
14	Service Station	-

* Number in parenthesis is the no. of the establishment as shown in Table A-84.

analyzed to determine the mean, coefficient of variation and range of concentration measured for each category involved.

Results and Discussion--

Quantitative characterization--The volume of wastewater produced per day by each of the establishments is presented in Table A-86. Also shown is the period of days over which each individual volume measurement was made. Utilizing the available physical characteristics of each establishment, the flows were converted to volume/day/unit values as presented in Table A-87.

Qualitative characterization--The results of qualitative analyses were grouped according to category and subsequently analyzed. The mean, coefficient of variation and range of values determined for each category are presented in Table A-88. As shown, there is a significant difference between the concentration of the selected parameters between categories of establishments. However, within each category the variation of concentrations is reasonably low as evidenced by the coefficients of variation. These results appear to support the previous discussion and recommendation that the use of average concentrations of composite wastewater samples is acceptable as a characterization approach.

It should be noted that the wastewater generated by each establishment remained in the holding tank for varying periods of time prior to sampling. Also, the entire contents of the holding tank was not always pumped completely. As a result, the values identified for COD and suspended solids may be somewhat less than those in the fresh, raw wastewater.

TABLE A-86. DAILY WASTE FLOW VOLUMES, L/day

Establishment					
No.	Type	Meas. Period*, Days	Data Pts.	Mean	95% C.I.
1	Bar/Tavern	8	45	2840†	2420 - 3290
2	Bar/Tavern	6	38	2080†	1700 - 2530
3	Campground	3†	101	23810	19660 - 27600
4	Church	31†	27	360†	290 - 450
5	Church	81†	8	300	210 - 380
6	Church	38†	21	290†	240 - 370
7	Church	43	16	270†	230 - 330
8	Country Club	8†	55	3360†	2530 - 4380
9	Country Club	5†	55	2650†	2230 - 3140
10	Country Club	3†	193	7820†	7030 - 8690
11	Drive-In Restaurant	8	108	2840†	2610 - 3140
12	Restaurant	3	234	5670†	5370 - 9750
13	School	23†	22	1470†	870 - 2570
14	Service Station	23†	26	930	760 - 1130
15	Service Station	4†	141	4910†	4350 - 5590
16	Service Station	9†	63	2190†	1890 - 2530
17	Service Station	5	148	4540†	4120 - 4990
18	Service Station	8	122	2340†	2150 - 2570

* Average period between holding tank pumpage. Includes non-working single days, but no off-season.

† Log-normalized data.

TABLE A-87. DAILY WASTE FLOW, L/day/unit

Establishment				
No.	Type	Unit	Mean	95% C.I.
1	Bar	Seats	57	49 - 64
2	Bar	Seats	19	15 - 23
3	Campground	Campsite	102	87 - 121
		Registration	300*	-
4	Church	Sunday Attendant	2.3	1.9 - 2.6
			(15.5†)	-
5	Church	Sunday Attendant	1.9	1.1 - 2.3
			(13†)	-
7	Church	Member	0.8	0.6 - 0.9
			(5.3†)	-
8	Country Club	Golfer	15*	-
10	Country Club	Golfer	64*	-
11	Drive-In Restaurant	Patron Car Space	36	33 - 39

(continued)

TABLE A-87 (continued)

Establishment				
No.	Type	Unit	Mean	95% C.I.
12	Restaurant	Seat	19	18 - 20
13	School	Student	9.1 (13‡)	5.3 - 16 -
14	Service Station	Bay	470	330 - 570
		Car Served	8.7*	-
15	Service Station	Bay	1630	1440 - 1850
16	Service Station	Bay	1100	950 - 1290
17	Service Station	Bay	2270	2060 - 2490
		Car Served	23*	-
18	Service Station	Bay	1170	1080 - 1290

* Values presented as L/unit.

† Assumes all flow on Sunday, value expressed as L/day/worshiper.

‡ Assumes all flow on Monday-Friday, value expressed as L/school day/student.

The results of this study have been combined with the results of the previous investigations to provide a comprehensive summary of water use/wastewater characterization data for the selected establishments. An evaluation of the data was conducted in an attempt to identify expected loadings from establishments within each category. If several pieces of data were available for a given parameter and establishment, a range of values was determined. Due to differences in establishment type and monitoring methods this was felt to be more appropriate than presenting a mean value. However, if a piece of data was felt to be unreliable due to the method of generation, it was omitted. The results of this evaluation are presented in Table A-89 for quantitative parameters, including mean daily flow and peak flow ratios. A similar summary table was not prepared for qualitative parameters due to a general lack of reliable information.

Summary

A substantial amount of characterization data presently exists. However, certain specific data are yet lacking or in need of verification. Additional extensive field studies are necessary, utilizing the recommended characterization approaches, to provide the missing data for establishments included in this study.

TABLE A-88. QUALITATIVE WASTEWATER CHARACTERISTICS, mg/L

Establishment Category	No. of Establishments Sampled	COD	TS	TVS	TSS	VSS	TKN	TP
Country Clubs	2	520(14)* 1.01 40-1820	1010(14) .34 650-1690	410(14) .65 120-1115	100(14) .62 26-195	70(14) .79 16-160	39(13) .70 2-67	12(10) .53 2-20
Churches	2	780(3) .39 512-1112	1850(3) .28 1470-2440	740(3) .23 560-890	590(3) .56 310-950	360(3) .71 210-660	48(3) .08 43-51	5(2) .76 2.1-7.0
Restaurants	3	1870(7) .50 1110-3450	2320(7) .35 1290-3580	1100(7) .34 770-1650	780(5) .85 480-1970	600(5) .94 210-1590	60(6) .50 9-98	40(3) .27 35-55
Schools	1	500(1)	2190(1)	450(1)	130(1)	60(1)	28(1)	-
Service Stations	6	360(20) 172 34-1140	1180(19) .35 460-2110	420(19) .60 120-1070	120(20) .92 22-260	80(20) .90 8-290	60(16) .71 4-175	7.4(13) .39 3-12.5

* Each data block includes: Mean (data points)
Coefficient of Variation
Range

TABLE A-89. QUANTITATIVE WASTEWATER CHARACTERISTICS SUMMARY

Establishment Category	Unit	Range of Mean Daily (No. of Data Pts.)	Peak Flow	
			Maximum Day/Mean Day	Peak Hour/Mean Day
Bar/Tavern	Seat	5-15 (1)[‡]	-	-
Bowling Alleys	Alley	49-56 (1)[‡]	-	-
Campgrounds	Camper	40 (1)	-	-
Picnic Parks	Picnicker	5 (1)	-	-
Churches	Member	.20 (3)	4.4	24.0
Golf Clubs	Golfer	4-16.9 (1)[*]	1.2	1.9[‡]
Laundromats	Machine	170-240 (3)	3.0	-
Motels	Patron Space	70	2.1	4.5, 6.9
Restaurants	Meal	6.4-11.7 (3)	1.4	2.0
Schools I[†]	Pupil	5-6(2)	2.4	-
Schools II[†]	Pupil	6.2-7.5 (4)	1.8, 1.4, 1.1	3.8
Schools III[†]	Pupil	8.2-19.9 (4)	1.1, 2.3, 3.1	3.2
Service Stations	Bay	290-600 (4)	3.7	26.5
Shopping Centers	1000 ft^2	164-215	1.4	2.0, 2.0, 2.5
Sports Facilities	Spectator	1.2[‡]	-	7.5
Theaters	Seat	2.7[‡]	-	-

[*] High value due to 700 seat restaurant contributions.

[†] School categories: I - restrooms only, II - restrooms and cafeteria, III - restrooms, cafeteria and showers.

[‡] Verification desirable.

METHODS FOR IN-HOUSE ALTERATION OF WASTEWATER CHARACTERISTICS

The characteristics of a wastewater can have a major impact on the design and performance of any wastewater treatment and disposal system. Recognizing this fact, increasingly more emphasis is being directed toward developing methods for in-house alteration of the raw wastewater characteristics. Elimination of potential pollutants at the source, such as flow, oxygen-demanding substances, suspended solids, nutrients, and pathogenic organisms, or their isolation in a concentrated waste stream could enhance conventional disposal methods or facilitate the development and use of innovative alternatives.

Consideration of the results of the previously discussed characterization work readily indicates that two powerful strategies for altering typical wastewater characteristics are (1) waste flow reduction and (2) waste segregation. A discussion of each strategy follows. It should be noted that although the primary thrust of the following discussions has been directed toward residential dwellings, many of the concepts and techniques presented have application to non-residential establishments as well.

Waste Flow Reduction

Methods to Achieve Waste Flow Reduction--

Waste flow reduction involves reducing the volume of water used within a household or other establishment, thereby producing a reduction in the volume of wastewater produced. To accomplish this, an array of concepts, techniques and devices have been proposed. To illustrate the extent and diversity of these methods, Table A-90 has been prepared. As shown, waste flow reduction methods may be divided into three major categories: (1) elimination of non-functional water use, (2) water-saving fixtures, devices and appliances and (3) wastewater recycle/reuse systems.

TABLE A-90. WASTE FLOW REDUCTION METHODS

I. Elimination of Non-Functional Water Use

  A. Eliminate Wasteful, Water-Use Habits
  B. Maintain Non-Excessive Water Pressure in House Supply
  C. Maintain Plumbing in Good Repair

II. Water-Saving Devices, Fixtures and Appliances

  A. Toilet
    1. Conventional toilet, tank inserts
    2. Dual-flush toilet devices
    3. Shallow-trap flush toilet
    4. Mechanically-assisted, low-volume flush toilet
      a. Compressed air
      b. Vacuum
      c. Grinder

(continued)

TABLE A-90 (continued)

    5. Non-water carriage toilets
        a. Composting
        b. Incinerator
        c. Closed-loop recycle (water or oil)
  - B. Bath/Shower
    1. Shower flow controls
    2. Reduced-flow showerhead
    3. Pressure-balanced mixing valve
    4. Air-assisted low-flow shower system
  - C. Clotheswasher
    1. Adjustable cycle settings
    2. Suds-saver feature
  - D. Dishwasher
    1. Adjustable cycle settings
  - E. Sink Faucets
    1. Faucet inserts
    2. Reduced-flow faucet valves
    3. Faucet aerators
    4. Pressure-balanced mixing valve

III. Wastewater Recycle/Reuse System

  - A. Bath/Laundry Wastewater Reuse for Toilet Flushing
  - B. Bath/Laundry Wastewater Reuse for Exterior Uses
  - C. Combined Wastewater Reuse for Toilet Flushing

Elimination of non-functional water use--Non-functional water use is typically the result of (1) wasteful, water-use habits, (2) excessive pressure in the house supply, and (3) inadequate plumbing and appliance maintenance. Wasteful water use habits, such as using a toilet flush to dispose of a cigarette butt or operating a clotheswasher or dishwasher with only a partial load, produce needless quantities of waste flow. Excessively high water pressure in the house supply can result in unnecessarily high water flow rates through faucets, showerheads and similar fixtures. Unseen or apparently insignificant leaks from household fixtures and appliances can generate substantial quantities of waste flow. Obviously, the potential for waste flow reduction through elimination of these types of non-functional water uses varies tremendously between homes depending on existing characteristics.

Water-saving devices, fixtures and appliances--To reduce the quantity of water used by a given household fixture or appliance in accomplishing a given task, conventional fixtures are being redesigned and new and innovative devices, fixtures and appliances are under development. Among these are toilet-tank inserts, faucet aerators, showerhead inserts, flow-regulated showerheads and faucets, water-saving dishwashers and clotheswashing machines and innovative toilets, including reduced flush and non-water carriage systems. Since toilet flushing, bathing and clotheswashing collectively account for over 70% of a typical household's waste flow volume, efforts to achieve major reductions have logically been concentrated in these three areas.

Wastewater recycle/reuse systems--A final method for reducing waste flow volumes involves processing all household wastewater or the fractions produced by certain activities for subsequent reuse. The flow sheets suggested have been numerous and vary considerably. However, major emphasis has been placed on recycling bath and laundry wastewaters through some form of treatment process for use in flushing water-carriage toilets and lawn irrigation.

Evaluation of Flow Reduction Methods--

A limited number of investigations have been conducted to identify the actual waste flow reductions possible through utilization of certain of the previously described methods. A brief review of several studies follows.

In 1969, Bailey, et.al., reported the results of a literature survey of methods for waste flow reduction from households conducted by the Electric Boat Division of General Dynamics. Flow reductions were projected for various water-saving devices used in a hypothetical household. To enable these projections, base water use was estimated at 210 L/day (55 gal/day) for household uses including 57 (15) for dishwashing, 130 (35) for laundry, and 19 (5) for cleaning, and 190 L/cap/day (50 gal/cap/day) for personal uses including 11 (3) for drinking and cooking, 78 (20) for bathing, 8 (2) for oral hygiene and 94 (25) for toilet flushing. The waste flow reductions predicted were,

Flow control shower heads - 23 L/cap/day
Faucet aerator - 1.9 L/cap/day
Shallow-trap toilets - 28 L/cap/day
Dual-flush cycle toilet - 66 L/cap/day
Washing machine sudsaver - 4.5 L/cap/day

In 1971, a follow-up study supported by the U.S. EPA was performed by General Dynamics to field evaluate certain promising methods identified in their first study. Eight households were included in the demonstration project, six in southeastern Connecticut and two in San Diego, California. At each of the homes, water meters were installed in the house supply line, as well as in the individual lines to the toilet, bath/shower, and laundry (five homes only). The water usage was recorded by the homeowners on a weekly basis. To provide base flow data, water use was monitored for six months prior to device installation. Then, flow reducing devices and recycle systems were installed, including shallow-trap toilets, two types of dual-flush toilet devices, flow-reducing showerheads, and a wastewater recycle system, and water was monitored for one year. After this period, the devices were removed and water use was monitored for another six months. A summary of the waste flow reductions produced by the devices is presented in Table A-91.

Flow-reducing showerheads were found to produce only a minor reduction in the total household flow. The investigators believed this to be due to the fact that the residents preferred baths over showers. The shallow-trap toilets produced a moderate reduction of about 3% to 9% of the total household flow. As expected, the most substantial water-savings (26%) were achieved with the recycle systems which processed the bath and laundry wastewaters for toilet flushing and lawn sprinkling.

TABLE A-91. WASTE FLOW REDUCTION SUMMARY
(Cohen and Wallman, 1974)

Device	Description or Type	Units Tested	Average Occupants	Water Savings	
				L/cap/day	% of Total
Flow Limiting Shower Head	9.5 L/min 13.3 L/min	3 8	5.8 3.8	1.6 3.7	0.8% 1.2%
Shallow-Trap Toilets	9.5-12.5 L/flush	6	4.5	14.8	6.9%
Dual Flush Toilet Devices	8/17 L/flush 14/17 L/flush	4 4	2.8 3.2	20.5 12.4	8.6% 3.3%
Wash Water Recycle System	Storage → Filtration → Cl_2 → Toilet Flushing/ Sprinkling	3	6.3	44.	26. %

This investigation also included two important aspects of waste flow reduction, namely, homeowner acceptance of the devices utilized and the net costs incurred through their use. Homeowner acceptance was evaluated through distribution of a formal questionnaire to the 16 adult occupants of the eight study homes. A summary of the questionnaire results in terms of user acceptance is shown in Table A-92. As shown, there was a clear indication of these homeowners' acceptance of the waste flow reduction devices utilized, with the possible exception of utilizing recycled wastewater for lawn watering.

TABLE A-92. HOMEOWNER ACCEPTANCE OF FLOW REDUCTION DEVICES
(Cohen and Wallman, 1974)

Device or System Tested	Number of Respondents	Percent of Responses Indicating Acceptance
Flow Limiting Shower Heads	16	88%
Shallow Trap Toilets	12	83%
Dual Flush Devices	6	100%
Wash Water Recycle for Toilet Use	6	67%
Wash Water Recycle for Lawn Sprinkling	4	50%

The costs for installation and operation of the various waste flow reduction devices was computed and the net savings or costs incurred were estimated based on typical water and sewer rates in the study areas. Although such calculations are of questionable validity due to rate structure mechanics, the investigators perceived the devices to be economically justifiable.

In 1971, the Washington Suburban Sanitary Commission (WSSC) began actively promoting public water conservation in an attempt to reduce wastewater flow volumes (Bishop, 1975). A major phase of WSSC's program involved the evaluation of pressure reducing valves, toilet tank inserts and flow-reducing shower heads. The evaluation involved the installation of 4,800 devices in 2,400 residential units; approximately one-half were single-family dwellings with the other half apartments and townhouses. In addition to the water-saving devices, a booklet containing suggestions for water conservation was distributed to each participant. Based on the results of one year of operation with the water saving devices, the apartment units and single family units had reduced their consumption by 12 and 18-20 percent, respectively, through use of the toilet tank inserts (weighted, water-filled plastic bottle). Flow-reducing shower heads (11.3 L/min) yielded a 12% reduction in consumption. It should be noted that a major portion of these water savings may have been due to improved water-use habits as a result of the information booklet distributed.

Follow-up studies by the WSSC included the installation of showerhead inserts in 100 townhouses (Bishop, 1975). Based on water-use monitoring over a ten-month period, daily water use was found to increase by about 2.5%. The investigators attributed the increase and the contradictory results of the two studies to user lifestyle habits. The lack of a supervised installation program in the latter study was also felt to be responsible for the water-use increase.

As part of the effort to characterize household wastewater conducted as a phase of this study, event water use was monitored at eleven homes for a total of 434 days. To assess the potential for waste-flow reduction in these homes, four water-saving methods were hypothetically applied to each home. These methods included: (1) reducing the toilet flush to 11 L (3 gal); (2) replacing the existing clotheswasher with a sudsaver model; (3) controlling the volume of water used for bathing to 57 L (15 gal) per bath or shower, and (4) recycling the bathing and laundry water for toilet flushing. The calculated flow reductions possible through utilization of these four methods are shown in Table A-93.

Utilizing an 11 L/flush (3 gal/flush) toilet, a sudsaver washer, and a 57 L (15 gal) bath/shower, waste flow reductions of 7 to 23% were calculated with an average of 17%. Recycling the bath and laundry wastewaters for toilet flushing could increase these reductions to 23 to 43% with an average of 33%. It should be noted that these reductions were calculated assuming that the number of events/cap/day remained unchanged after application of the water-saving methods.

A comparison of these results with those of the previously discussed studies indicated some disagreement as to the water savings and waste flow reductions achievable through utilization of a particular device. This is as expected, however, in light of the variation in water-use habits between individuals and homes. For example, if the residents of a given household prefer baths for bathing, a flow-reducing showerhead will be of little value. Similarly, a low-flush toilet will not have as significant an effect for the household where the residents are very mobile and tend to use restroom facities outside the home. Due to these types of user habit variations, it is

TABLE A-93. PROJECTED FLOW REDUCTION SUMMARY

		L/cap/day with Method Shown				% Reductions		
		(1)	(2)	(3)	(4)			
Family Unit	Mean Meas. L/cap/day	11 L/Flush Toilet	Sudsaver-Washer-105 L/use	57 L Bath/Shower	Recycle System	Using (1)-(3)	Using (4)	Using (1)-(4)
A	214	204	205	188	191	22	11	33
B	96	89	96	88	70	16	27	43
C	147	145	135	137	128	16	13	29
D	155	155	151	142	124	11	21	32
E	157	145	139	150	137	23	13	36
F	128	120	128	121	112	12	12	24
G	113	105	113	106	96	7	15	22
H	188	179	179	186	162	10	14	24
I	158	147	158	150	139	11	12	24
J	171	154	163	156	135	23	19	43
K	215	207	202	202	182	16	15	31
Average (Weighted)	161	152	154	150	135	17	16	33

very difficult to accurately predict the actual waste flow reduction which may result from use of a particular device in a given household. One must exercise caution in applying the laboratory performance data or even limited field information for a given water-saving device to an "average" household water use, to yield a particular flow reduction for a given household.

Waste Segregation

A second major technique for altering the characteristics of household wastewater involves separating the toilet wastes (black water) from the other household wastewaters (grey waters) for separate treatment and disposal. This segregation scheme also includes the elimination of the household garbage disposal. The basis for suggesting this segregation lies in the characteristics of the segregated waste streams.

The results of the characterization studies previously discussed in this appendix demonstrated that the use of a garbage disposal can increase the BOD_5 and suspended solids loading in household wastewater by 22% to 64% and 43% to 94%, while adding little flow, nitrogen or phosphorus (Table A-36). For this reason and the fact that most of the wastes handled by a garbage disposal can be handled effectively as solid wastes, the garbage disposal was eliminated as part of the segregation scheme.

The division of chemical/physical pollutants between the black water and grey water, are shown in Table A-94. On the average, the grey water contributes about 65% of the flow, 70% of the phosphorus and 63% of the BOD_5, while the black water contributes about 61% of the suspended solids, 82% of the nitrogen and 37% of the BOD_5.

TABLE A-94. BLACK AND GREY HOUSEHOLD WASTEWATER CHARACTERISTICS*

	Grey Water				Black Water			
Pollutant	Mean %	Range %	Mean g/c/d	Mean mg/L	Mean %	Range %	Mean g/c/d	Mean mg/L
BOD_5	63	51-80	28.5	255	37	20-49	16.7	280
Suspended Solids	39	23-64	17.2	155	61	36-77	27.0	450
Nitrogen	18	1-33	1.9	17	82	67-99	8.7	145
Phosphorus	70	58-86	2.8	25	30	14-42	1.2	20
Flow	65	53-81	111 L/cap/day		35	19-47	60 L/cap/day	

* Based on the results of studies by Olsson, Karlgren & Tullander, 1968; Cohen & Wallman, 1974; Ligman, Hutzler & Boyle, 1974; Laak, 1975; Bennett & Linstedt, 1975; and this study - garbage disposal results omitted.

The microbiological characteristics of the two waste streams, and in particular the grey water, are of prime importance. Both enteric and non-enteric organisms are of interest, and the obvious concern is for the potential occurrence of pathogenic organisms in the grey and/or black water streams. Obviously, a prerequisite to encountering any pathogenic organisms in either waste fraction is that a member of the household or visitor thereof, be shedding pathogens. Unfortunately, data concerning the percentage of the population shedding various potential pathogens is highly variable and, in many cases, altogether lacking.

In regard to enteric organisms, samples of the effluents from septic tanks receiving combined household wastewater have been found to consistently contain significant concentrations of indicator bacteria and the potential pathogen, Pseudomonas aeruginosa (McCoy and Ziebell, 1976). Staphylococcus aureus and salmonellae have also been isolated, but only infrequently and in much lower concentrations (McCoy and Ziebell, 1976). Intuitively, one would expect that the great majority of these organisms (predominantly enteric) in combined household wastewaters are contributed in the toilet wastes, with the grey water being comparatively less contaminated. The microbiological studies conducted as a part of this project indicate that bath and laundry wastewaters do possess a potential for yielding enteric contamination of the grey water, possibly including enteric pathogens. However, the degree of enteric contamination and the potential for encountering enteric pathogens in the grey water appears significantly lower than that of either the black water or combined household wastewater. Table A-95 illustrates this point.

In addition to the enteric organisms, non-enteric organisms such as those discharged in sputum and washed from the skin, deserve attention. If someone in a household were suffering from a respiratory or an external

TABLE A-95. ESTIMATED BACTERIOLOGICAL CHARACTERISTICS OF VARIOUS HOUSEHOLD WASTEWATER STREAMS, organisms/100 mL

Effluent Organism	Grey* Water	Toilet** Wastewater	Combined† Wastewater	Combined‡ Wastewater Septic Tank Effluent
Total Coliforms	1000	6,000,000	2,100,000	3,400,000
Fecal Coliforms	660	6,000,000	2,100,000	420,000
Fecal Streptococci	250	60,000	21,000	3,800

* Contribution of flow by bath and laundry approx. equal, average of means for two events calculated, contribution of organisms by other household events (kitchen sink, etc.) assumed equal to this average.

**Calculated assuming: 100 wet gm feces/cap/day and toilet flush flow of 60 L/cap/day with bacterial contents per wet gram of feces equal to, total and fecal coliforms = 10^7 and fecal streptococci = 10^5.

† Calculated: Value = .65 grey water value and .35 toilet wastes value.

‡ McCoy and Ziebell, 1976.

epidermal infection, the possibility would exist for the associated non-enteric organisms to enter the grey wastewater stream. This is evidenced by the results of analyses on coliform and streptococcal isolates of bath and laundry origin noted previously, which indicated that much of the bacterial contamination in the samples was from the natural environment or skin flora of man. To assess the magnitude of this potential for encountering non-enteric organisms in grey water and place it in perspective one has to consider the relative importance of grey water, even without treatment, as a transmission route compared to other routes, such as the vehicles, vectors and person-to-person contact encountered in normal daily activities. Consideration of this factor, as well as the added fact that many of the non-enteric organisms of concern are ubiquitous opportunists would seem to indicate that in general, transmission of non-enteric organisms through household grey waters is not of major concern.

Methods to Achieve Waste Segregation--

To provide for segregation and separate handling of the toilet wastes, several alternatives to the conventional flush toilet have been proposed. In a recent study, over a dozen alternative toilets were identified (Rybczynski and Ortega, 1975). Those which have been receiving increased promotion and attention include the composting, closed loop recycle, very low flush/holding tank systems, and incinerating toilet systems.

Evaluations of Segregated Treatment Systems--

Very little research has been conducted on segregated treatment systems. Obviously, waste segregation was an integral part of society for many years for those utilizing the pit privy. However, only recently have actual investigations of segregated waste treatment been initiated. For the most part, these investigations have been directed toward evaluation of the non-water carriage toilet systems. Typically, grey water disposal has been accomplished with a conventional septic tank-soil absorption system, possibly of reduced size due to a reduced hydraulic loading. However, other more innovative alternatives may be feasible due to the reduced pollutant load and contamination of the grey water.

Implications for Onsite Wastewater Disposal

Impact on the Operation of Existing Systems--

For an existing soil disposal system which is functioning satisfactorily, reducing the wastewater flow volume or eliminating pollutants at the source might serve to extend the life of the soil absorption system. However, by what factor, if at all, is presently unknown. If the existing system were failing, with wastewater daylighting or backing up, flow reduction efforts could reduce the waste load sufficiently to remedy the situation. For those establishments utilizing a holding tank system, waste flow reductions could yield significant savings in pumping charges.

With regard to existing disposal systems, it is important to realize that simple, relatively inexpensive waste flow reduction measures may yield significant benefits, especially to those households which have disposal systems which are hydraulicly overloaded. If a given technique or device does not yield the expected reduction, in many cases little time, effort and money will have been lost. If the objective was to remedy an overloaded system and waste flow reduction techniques proved unsuccessful, alternative, more costly solutions such as expanding or replacing the system, could then be tried.

Impact on the Design and Operation of New Systems--

Accounting for the altered waste loads provided by water conservation practices can be a complex task. One must be relatively confident that the use of a given device or system will yield a predicted waste load reduction.

Further, one must be confident that the technique or device utilized will be accepted by the present users as well as future users and be used throughout the service life of the disposal system. If a device or system does not yield an expected reduction or is disconnected or replaced, the waste loads to the onsite disposal system will be greater than expected, possibly resulting in system failure. If this happens, remedial actions could be taken, including upgrading or replacing the disposal system; however, this is often costly and very difficult to accomplish. If modifications are permitted in the design of an onsite waste disposal system due to an expected altered waste load, provisions should be made for alternative waste disposal methods.

Simply reducing the waste flow volume to a conventional soil disposal system should enable the size of the system to be reduced in proportion to the expected reduced flow. However, any reduced sizing should probably be restricted to the soil disposal field with any pretreatment process maintained largely full-size to provide the necessary capacity to treat and attenuate peak flows. In addition to a reduced flow, waste segregation practices provide for a reduction in the quantity and concentration of certain pollutants, a fact which has been suggested to render grey water more acceptable to soil absorption. However, efforts to correlate soil clogging with household wastewater characteristics have been generally unsuccessful, especially in the more problematic structured soils (Appendix B of this report). At present, any reduction in field sizing should be based solely on a reduced hydraulic loading.

Waste segregation practices appear to offer the potential to facilitate alternative systems, including surface disposal, exterior reuse, or modified subsurface disposal of household grey water. Detailed field investigations are necessary to adequately evaluate this potential.

PART 2

EVALUATION OF ONSITE WASTEWATER TREATMENT METHODS

INTRODUCTION

The choices available for wastewater disposal are numerous, yet only a selected few will prove to be both economically and environmentally acceptable. The selection process involves the evaluation of technical feasibility, cost effectiveness and administrative feasibility. Table A-96 lists some of the treatment processes potentially available for onsite treatment of wastewater. This portion of the report will briefly review many of these processes and subsequently discuss the results of laboratory and field experience with a number of promising systems installed by the University of Wisconsin research group.

TREATMENT METHODS

Biological Processes

Anaerobic Systems--

Anaerobic processes can be divided into two distinctive biochemical groups, the fermentations and the anaerobic respirations. Fermentations are defined as energy yielding metabolic processes in which organic compounds serve as both electron donors and electron acceptors. With anaerobic respirations, an oxidized inorganic compound such as nitrate, sulfate and carbonate, serves as the ultimate electron acceptor. Both fermentations and anaerobic respirations play an important role in waste treatment processes. In the anaerobic environment, bacteria will normally attack organic molecules such as lipids, cellulosic materials and proteins reducing them to a molecular size which can be subsequently fermented (hydrolysis). A complex group of anaerobic microbes will then ferment these molecules to acetate, other organic acids, alcohols, H_2, CO_2, NH_3 and sulfide. A second, still rather unknown group of bacteria convert the organic acids and alcohols to more acetate and H_2. Finally, a third group of diverse species of methanogenic bacteria which utilize H_2 and CO_2 and acetate will produce CH_4 and H_2. This sequence of reactions is most important in anaerobic treatment processes for it is the generation of methane gas (and small amounts of CO_2 and H_2) which result in the reduction of BOD in the wastewater. Hydrolytic and fermentative reactions up to the methanogenic step merely transform organic molecules but do not result in removal of BOD. Since the methanogenic organisms are very sensitive to environmental conditions, the effectiveness of anaerobic fermentative treatment processes is dependent to a great degree on process control.

TABLE A-96. POTENTIAL WASTEWATER TREATMENT PROCESSES - ONSITE DISPOSAL

- Biological Processes
  - Anaerobic
    - Septic Tanks
    - Fixed Media Systems
      - Sand/Granular
      - Synthetic Media
  - Aerobic
    - Suspended Growth
      - Activated Sludge
        - Batch
        - Continuous Flow
    - Fixed Media
      - Sand-Intermittent Filter
        - Expanded Bed
      - Coarse Media
        - Trickling Filters
        - Rotating Biological Media
        - Tray/Media Contactors
- Physical/Chemical Processes
  - Adsorption
  - Ion Exchange
  - Chemical Precipitation
  - Disinfection
    - Chlorine
    - Iodine
    - Ultraviolet
    - Ozone

The presence of sulfates, nitrates and other selected oxidized inorganics in wastewater is also of importance in anaerobic systems. The anaerobic respiration of nitrate leading to the ultimate production of nitrogen gas is used to achieve nitrogen removal in wastewater. The reduction of these oxidized compounds also serve to produce a stabilization of BOD.

The kinetics of anaerobic processes are dependent upon the substrate quality and quantity, the number and type of microbes and the environmental conditions within the process. Theoretical growth rates of anaerobic organisms are comparable to aerobes, but environmental conditions are often more critical to optimum activity of the anaerobes. Thus, for optimal methane production, no dissolved oxygen can be present, ORP values must be low, temperature ranges are best between 32-37°C or 50-55°C, and pH should be maintained between 6.7 and 7.4.

In short, anaerobic treatment has many advantages that make it attractive as a household wastewater treatment process. It requires no power input and will continue with no attention from the homeowner. Biological growth is so low that it is not necessary to remove sludge from the system very frequently. The effluent produced, however, can be malodorous and high in pathogenic organism indicators but, if disposed of in the subsurface soil, it need not be a nuisance.

Household Units--

Septic tank--The septic tank has been virtually the only means of treatment employed prior to final disposal in household waste treatment systems. Its design, installation, and operation is the simplest of all treatment methods devised and it is also the least expensive. Its performance has been in question because of the large number of failures in the attendant soil absorption field, but when installed properly, the septic tank - soil absorption system is one of the best methods of treatment and disposal for the single home.

The primary purpose of the septic tank is to protect the soil absorption field from becoming clogged by solids suspended in the raw wastewater. The septic tank provides three functions to accomplish this goal. First, it acts as a settling chamber to remove much of the settleable and floatable materials. Second, it provides storage for the solids removed and an anaerobic environment for their digestion by anaerobic organisms. The third function is the anaerobic treatment of the non-settleable particles to change their character from a gelatinous nature to a non-gelatinous nature to reduce the clogging potential of the solids remaining in suspension (Ludwig, 1950; Nottingham and Ludwig, 1948).

Though simple in principle, there are certain design features that must be included in every septic tank. The tank is generally sufficiently large to provide a hydraulic detention time of at least 24 hours at the expected average daily flow, after allowing two-thirds of the tank volume for sludge/scum storage. Inlet baffles are generally installed to prevent short circuiting of the liquid across the top of the tank and to mix the fresh sewage with the biologically active liquid and sludge in the tank. The invert of the outlet should be at a level sufficiently below the inlet invert to prevent backwater and stranding of solids in the sewer line during momentary rises in the tank level when wastewater is intermittently discharged from the home. Baffling of the outlet or use of an outlet tee is required to prevent the scum from leaving with the effluent. Venting through the house vent is necessary to allow the release of gases produced. Depth and shape of the tank are generally specified within limits to minimize any effect on treatment efficiency within reasonable limits. Recommended capacities and dimensions can be found in the Manual of Septic Tank Practice (1967) which incorporates many of the findings from a five-year study of septic tank systems by the U.S. Public Health Service.

Many investigators have attempted to improve the quality of effluent produced by septic tanks and to reduce its variability. Their studies have resulted in recommended design modifications which primarily reduce the amount of suspended solids that are discharged in the effluent. Ludwig (1950) found elongated tanks with length to width ratios of 3 to 1 and greater, retarded short circuiting through the tank and improved suspended solids removal. He and others have found that two or more tanks in series or a compartmentalized tank provides better treatment (Ludwig, 1949, 1950; Weibel, et al., 1949, 1954). The Manual of Septic Tank Practice (1967) recommends a two compartment system with the first compartment 1/2 to 2/3 the total volume. Each compartment must be vented through the household plumbing and manholes provided for access.

Another modification is the gas deflection baffle (Baumann and Babbitt, 1954). Rising gases from anaerobic metabolism within the sludge disturbs the tank contents so that solids are resuspended and washed out of the tank. It also appears that the tank "overturns" each spring and fall in response to seasonal temperature changes, causing an increase in the concentration of solids discharged. Conventional effluent baffles are unable to prevent this washout.

Another concern of investigators has been the effect of household chemicals on the biological treatment provided by the tank. All studies performed indicate that detergents, soaps, bleaches, drain cleaners, water softening brine and other such materials have no adverse effect as normally used (Fuller, 1952; Truesdale and Mann, 1968; Weibel, 1955b). Septic tank operation is not found to be improved and, in fact, may be harmed by the addition of disinfectants or other chemicals marketed for such purposes (Weibel, 1955b). Enzyme additives have not proven beneficial. Generally, chemical additions are not recommended.

Maintenance requirements for septic tanks are minimal but they must be performed regularly to prevent the soil field from clogging. Once each year the sludge and scum levels should be checked by a procedure similar to the one outlined in the Manual of Septic Tank Practice (1967). If sludge or scum accumulations are excessive, the tank should be pumped to remove them before solids are discharged to the field. Anaerobic digestion of the sludge will reduce the volume of accumulations, but a maximum of 40 percent reduction could be anticipated under the most ideal conditions (Truesdale and Mann, 1968). Nondegradable solids will accumulate and these must be removed. Generally it is good practice to pump the tank once every three years dependent upon use.

Anaerobic contact tanks--Though some anaerobic treatment of the liquid fraction of the waste occurs within the septic tank, the majority of the soluble organics pass through the system because they are not brought into contact with the anaerobic organisms. If the bacteria were mixed with the liquid in some way, much better removals of BOD would be realized. This is known as the anaerobic contact process.

The anaerobic contact process can take one of two forms. Either the sludge can be anaerobically mixed with the waste which is analagous to the activated sludge process, or the liquid can be passed through porous media upon which the bacteria cling, the anaerobic counterpart to a trickling filter. Both configurations have been investigated, but rarely applied to household wastewater treatment.

Coulter, et al., 1957, first proposed the anaerobic contact process on a small scale for treating wastes from a small subdivision. Pilot plant studies revealed that upflow of raw wastewater through the sludge blanket and use of slow mixing to break up and disperse sludge solids produced BOD_5 and suspended solids removals of 50-65% and 84% respectively after 18 hours contact. Full scale studies for an anaerobic contact system for a 350 home subdivision produced an average of 33.6% BOD_5 removal and 76.8% removal of suspended solids. The system was very stable and required little attention. The disadvantage of this system was, primarily, its inability to handle hydraulic surges.

The anaerobic filter appears to be a more practical system for small flows. Upflow of wastewater through a variety of porous media have proven successful for treating wastewater (Young and McCarty, 1969; Jeris, 1975).

Such a filter has been tried in India, following septic tanks that received toilet wastes only (Raman and Chakladar, 1972). It was designed for loading rates slightly higher than a low rate trickling filter. An effluent quality was sought that could be suitable for surface discharge where subsurface disposal was not possible. Three such systems of different configurations were installed and compared. Two upflow and one downflow-upflow configurations were tried. The total volume of each filter was 1/3 to 1/4 the volume of the septic tank served. Crushed brick and stone chips (1.3-2.0 cm) were used as media to depths of 38 to 85 cm (1.25 to 2.75 ft).

The filters performed well at an application rate of 4.1 to 6.1 cm/d (1 to 1.5 gal/ft^2/day). The filter effluent was free from odor, clear or translucent and had a light or pale yellow color. Results of the studies appear in Table A-97..

TABLE A-97. RESULTS OF EFFLUENT ANALYSES FROM AN ANAEROBIC FILTER FOLLOWING A SEPTIC TANK (Raman & Chakladar, 1972)

Waste Fraction	Septic Tank Effluent Concentration	Filter Effluent Concentration	Percent Removal
BOD_5	170 - 240 mg/L	35 - 70 mg/L	65 - 75%
SS	350 - 450 mg/L	150 - 190 mg/L	53 - 60%
Turbidity	200 - 400 JTU	20 - 60 JTU	-

Clogging did occur with these filters after nearly two years of operation. They were cleared by opening a plug in the distribution pipe to reverse the direction of flow and flushing from the top with 56 liter (15 gal) of water. A 2.5 to 15 cm (1 to 6 in) headloss was experienced throughout the runs.

Denitrification reactors--Biological denitrification has been extensively evaluated as a means of removing nitrogen from municipal and industrial wastewaters and irrigation return flows (Eliassen and Tchobanoglous, 1968; Tamblyn and Sword, 1969; St. Amant and Beck, 1970; McCarty and Haug, 1971; Savage, 1974; Francis and Callahan, 1975). This approach, when used with large scale accessible reactors, appears to be technically feasible and economically competitive with other nitrogen removal methods.

Bioenergetics of denitrification--The main gaseous N products from biological denitrification of NO_3^- and NO_2^- are N_2O and N_2 (Alexander, 1961). Nitrous oxide (N_2O) is further reduced to N_2 in aqueous systems because of its high water solubility and thus is not a significant end product in reactor systems (McCarty and Haug, 1971). The process is essentially a respiratory mechanism in which NO_3^- or NO_2^- replaces O_2 as the terminal electron acceptor. It is carried out by a wide variety of ubiquitous facultative heterotrophic bacteria, and by certain autotrophs. It involves a change in the average valence level of N from +5 (NO_3^-) to 0 (N_2). The precise pathway remains unclear, but NO_2^- (N^{+3}) and $N_2O(^{+}1)$ are definite intermediates. The reaction is often written using CH_3OH (methanol) as the electron donor (eq. 21 to 23).

$$NO_3^- + 1/3CH_3OH \rightarrow NO_2^- + 1/3CO_2 + 2/3H_2O \quad (21)$$

$$NO_2^- + 1/2CH_3OH \rightarrow 1/2N_2 + 1/2CO_2 + 1/2H_2O + OH^- \quad (22)$$

Sum:

$$NO_3^- + 5/6CH_3OH \rightarrow 1/2N_2 + 5/6CO_2 + 7/6H_2O + OH^- \quad (23)$$

Thus, 1.9 g of CH_3OH are oxidized per g of NO_3^- nitrogen. However, additional CH_3OH (or any other available organic compound which is serving as an electron donor) is needed to consume O_2 (if present) and provide substrate for microbial growth. About 30 to 40% extra carbon substrate than theoretical is usually required, providing the C substrate is soluble and readily available (McCarty, et al., 1969).

Anoxic conditions are also a prerequisite for biological denitrification. The pH and temperature of the system affects the rate of denitrification. Most studies indicate optimal denitrification at temperatures between 20 and 25°C, and pH of 7 to 8 (Alexander, 1961; Broadbent and Clark, 1965). McCarty, et al., (1969) evaluated acetone, methanol, ethanol and acetate as energy sources, and found that the rates of denitrification were not appreciably different from methanol provided sufficient time for acclimation to methanol was permitted. More details on this process can be found in the U.S. EPA Manual for Nitrogen Control (1975).

Dissimilatory nitrate removal can also be accomplished with the ubiquitous chemolithotroph Thiobacillus denitrificans which is capable of oxidizing elemental sulfur under anaerobic conditions, using nitrate as the electron acceptor:

$$5S + 6KNO_3 + 2H_2O \rightarrow 3K_2SO_4 + 2H_2SO_4 + 3N_2 \quad (24)$$

If $CaCO_3$ is used to buffer this system the reaction would be:

$$5S + 6KNO_3 + 2CaCO_3 \rightarrow 3K_2SO_4 + 2CaSO_4 + 2CO_2 + 3N_2 \quad (25)$$

Types of denitrification reactors--Several types of biological denitrification units have been evaluated. These include modified activated sludge units, packed bed reactors, anaerobic columns, and anaerobic ponds.

The modified activated sludge approach involves recycling of sludge from a settling tank to maintain adequate bacterial populations (Francis and Callahan, 1975). It is subject to several problems, including high rates of assimilatory nitrate reduction ($NO_3^- \rightarrow$ organic N), long residence times, and excessive washout of microbial biomass. Rotating biological discs can be operated anaerobically, and appear to do an excellent job of denitrification (Davies and Pretorius, 1975).

Packed bed or packed column reactors have been found to be quite suited for biological denitrification. These units are enclosed reactors which contain submerged inert packed material. Support materials have included sand, gravel, coal, activated carbon, plexiglas, and polyethylene (Requa and Schroeder, 1975; Savage, 1974; Davenport, et al., 1975; Francis and Callahan, 1975). Problems include high head loss, clogging and short circuiting. However, these can be overcome with proper design.

Erickson, et al. (1971), studied a system to remove N and P from livestock wastes. This system used a limestone bed to remove P. Anaerobic conditions were maintained in the bed, and molasses added as an energy source for denitrification. Their initial results were encouraging.

Kinetics--A number of steady-state kinetic models for denitrification have been proposed (Francis and Callahan, 1975). These models assume sufficient inorganic nutrients are present and that either the electron donor or nitrate limits the growth rate. Monod (1950) kinetics have been found to adequately describe the kinetics of nitrate removal (Requa and Schroeder, 1969), but requires valid estimates of biomass and concentrations at various times, which is usually extremely difficult to obtain. Other kinetic models which have been used include zero order (Doner, et al., 1974) and first order (Francis and Callahan, 1975).

Aerobic Systems--

The most efficient biological method of reducing the organic content of dilute liquid waste is by aerobic treatment processes. Basically, the organisms responsible for treatment possess the ability to decompose complex organic compounds and to use the energy liberated for reproduction and growth. That part of the organic matter used to produce energy is converted to essentially stable end products of carbon dioxide, water, and ammonia, while the remainder is converted into new cells which are subsequently settled and removed from the liquid before the waste is discharged into the receiving environment. Oxygen must be continuously supplied during the aerobic process, since it acts as the final electron acceptor for oxidation of organic matter. It is during this electron transfer that there is liberation of large amounts of energy used for the synthesis of new cells. The quantity of oxygen required to stabilize organic matter depends on the organic content of the waste, and the physiological condition of the organisms.

In conventional aerobic treatment processes the synthesis of new biological growth can be expected to range between 50 and 60% of the dry weight of organic matter fed to the unit. As the cell residence time increases, the amount of cell material that is synthesized decreases. However, in no system yet conceived is there a total reduction of sludge in the process owing to the presence of inert material in the raw wastewater as well as to the synthesis of non-degradable solids. Solids wasting schedules from aerobic processes are dependent upon mass transfer capacity of the reactor as well as solids handling capacity of the separator.

A wide variety of kinetic models have been developed for aerobic processes in the treatment of wastewaters. The rates at which organic matter or other substrates are stabilized in aerobic systems are dependent upon a number of factors including the waste characteristics, pH, temperature, flow regime, and presence of toxic materials. Monod (1950) kinetics have been found to adequately describe the rates of most aerobic processes, although a number of modifications of this kinetic model appear in the literature.

Several types of aerobic biological systems have been developed over the past seventy years. These include suspended growth systems in which the microbial cells are held in suspension and the organisms are in intimate contact with the wastewater. Oxygen is normally transferred to the waste either through diffused air or mechanical methods of gas transfer. The other major type of biological system is made up of fixed film arrangements in which the waste passes over a film of microorganisms attached to some type of media. The media may pass through the waste, or the waste may pass over the media, depending upon the system design.

Household Units--

Despite their advantageous treatment capabilities, use of aerobic pretreatment units in private home systems is generally discouraged by health officials because of their high susceptibility to upsets. Without regular supervision and maintenance, the aerobic unit may quickly lose its efficiency. Since the homeowner cannot be relied upon to pump his septic tank regularly, most health officials feel that the homeowner also would not perform the added maintenance required by aerobic units (Voell and Vance, 1974; Waldorf, 1977).

To avoid this problem, manufacturers have tried several processes and have incorporated various design features into the design of treatment units in an attempt to reduce the need for constant surveillance. At least three aerobic process flow schemes are presently on the market in the low flow waste treatment field. They are extended aeration, trickling filtration, and rotating biological disks. Each process has its own unique operational characteristics and design features which must be employed to maintain a high effluent quality.

Extended Aeration--Extended aeration processes are modifications of the activated sludge process employing long sludge ages and detention times. Details of various process flowsheets used may be found in the literature (National Sanitation Foundation, 1966; Howe, 1961; and Pillai, et al., 1971).

It appears that extended aeration units are a good alternative to the septic tank if certain operational problems could be solved. One of the major difficulties is periodic discharge of large amounts of solids caused by excessive accumulation of mixed liquor solids, shock hydraulic loadings, and denitrification in the final clarifier (Howe, 1961; Pillai, et al., 1971; Reid, 1971; Bennett and Linstedt, 1975).

Other problems which frequently occur are inadequate control of the air supply, foaming in the aeration tank, clogging of sludge return lines, excessive solids return rates, and grease build-up (Howe, 1961). Therefore, careful design and operation of these units are required to maintain a high quality effluent. This is particularly true with units designed for the single household which must run for long periods of time with no attention while receiving intermittent and highly variable waste flows.

A successful household waste treatment unit must produce and maintain a high quality effluent regardless of changes in waste characteristics and flow, yet, accomplish this with few mechanical parts and little operational attention. A number of design features incorporated in extended aeration units are briefly discussed below.

1. *Process Flow--Continuous Versus Batch Operation*

   Continuous flow-through and fill-and-draw or batch operations are used in household waste treatment units. The continuous flow-through units discharge effluent continuously while receiving raw waste. The batch, on the other hand, collects the raw waste over a period of time, treats it throughout the holding period and discharges the treated waste only at the end of the collection period. Advantages and disadvantages are given in Table A-98.

2. *Greases and Gross Solids Handling*

   Effective grease and gross solids handling is important in maintaining overall treatment efficiency. If greases and inert objects are not removed or broken up, they will accumulate in the system. Several handling methods are employed including primary settling (septic tank), screens, and "hydraulic comminutors" which rely on hydraulic turbulence to break up large solids (Table A-99).

3. *Final Clarification*

   The final clarification step determines the overall efficiency of the unit. Since nearly complete stabilization of the influent BOD is achieved through the plant, the effluent BOD is due primarily to suspended biological solids which were not settled out.

   Final clarification must provide effective liquid-solids separation, effective sludge return, and floating solids removal. Both gravity settling and filtration are employed. Gravity separation is most frequently used. Quiescent settling, upflow clarification, and plate settlers are used together or separately in chambers following the aeration tank. Solids separation by filtration may also be employed. The treated wastewater passes through the filter prior to being discharged thereby retaining the solids in the system. While filtration produces a superior effluent it is very susceptible to clogging. Various separation techniques are summarized in Table A-100.

TABLE A-98. ADVANTAGES AND DISADVANTAGES TO PROCESS FLOW - EXTENDED AERATION

Process Flow	Advantages	Disadvantages
Continuous	1. Can accept wastes at all times 2. Requires fewer mechanical parts	1. Hydraulic surges can scour out sludge solids 2. Short circuiting of flow possible 3. Uncontrolled, continuous gravity discharge
Batch	1. Prevents solids carry-over from hydraulic surges (except at pump out) 2. Prevents short circuiting of waste 3. Controlled pumped discharge 4. Requires no sludge return facilities	1. Requires submersible pump 2. Should not accept wastes during settling and pumping unless a holding tank is employed

TABLE A-99. ADVANTAGES AND DISADVANTAGES TO GREASE/
GROSS SOLIDS HANDLING - EXTENDED AERATION

Process	Advantages	Disadvantages
Primary Settling (Anaerobic Chamber)	1. Removes and/or partially degrades large objects and solids before further treatment 2. Removes and hydrolyses greases 3. Increase total volume and number of chambers of treatment unit 4. Can be added to any unit	1. Increases size and cost of treatment unit 2. Offensive odors may be released in aerobic process 3. Periodic pumping and disposal of accumulated sludge required
Screens	1. Removes large objects and rags 2. Does not increase size and cost of treatment unit significantly	1. Requires frequent cleaning and disposal of debris or clogging will occur 2. Does not remove greases
Hydraulic Comminutors	1. Breaks up large solids for further treatment 2. Does not increase size and cost of treatment unit significantly	1. Does not remove greases 2. Does not remove or break up large inert objects

TABLE A-100. ADVANTAGES AND DISADVANTAGES TO SEPARATION PROCESSES - EXTENDED AERATION

Separation Method	Advantages	Disadvantages
Gravity Settling (Batch)	1. Complete quiescence during settling 2. No skimming of floating solids or sludge return facilities required	1. Influent waste may enter, disturbing settling phase unless additional tank is used
Settling Chambers (Continuous Flow)	1. Simple and relatively troublefree	1. Floatables must be removed ahead of unit 2. Energy from the aeration chamber may be transferred to the clarifier creating turbulence and upset
Plate Settlers	1. Increased rate of settling reducing discharges of solids	1. Solids build up on plates requiring frequent spraying for cleaning
Upflow	1. Better solids removal through sludge blanket	1. Sludge blanket very susceptible to hydraulic shock loading 2. Floatables must be removed ahead of unit
Filtration	1. Superior solids removal including floatables 2. No skimming of floating solids or sludge return facilities required	1. Filter cloth clogging is major problem 2. High head loss through filter as it clogs

4. *Sludge Return*

An effective sludge return is vital in providing biological seed for the aerobic process. Air lift pumps, draft tubes working off the aerator and gravity return methods are used. Batch processes or those that use filters for the separation of solids require no provision for sludge return. Rapid sludge removal from the final clarifier is beneficial to prevent the development of anaerobic conditions which are ideal for denitrifying bacteria. The resulting nitrogen gas produced from the biological conversion of nitrate will float the sludge. Unless means are provided for skimming these solids off and returning them to the aeration tank they can be lost to the treatment process. Too rapid a rate of sludge return, however, can increase the overflow rate of the clarifier to the point where the sludge does not settle adequately.

TABLE A-101. ADVANTAGES AND DISADVANTAGES TO SLUDGE RETURN METHOD - EXTENDED AERATION

Sludge Return Method	Advantages	Disadvantages
Gravity	1. No mechanical parts	1. Bridging of sludge may occur
Air-lift Pumps	1. Provides positive return 2. Can be run off of air supply for aeration	1. May clog
Draft Tube	1. Provides positive sludge return 2. Can be activated by aerator	1. May clog
Batch Process	1. No mechanical parts	1. Solids may be pumped out if sludge settles poorly or sludge blanket is high due to infrequent pumping

Floating sludge is ignored by the majority of manufacturers. With no means of return, solids carry-over can become a serious problem. If the unit has not received wastewater for a long period of time while under aeration, much of the sludge may float and be lost. Table A-101 compares sludge return systems.

Trickling filters--The trickling filter has been widely used in secondary treatment of municipal wastes for many years. Its low cost of installation and its relatively maintenance-free operation explain its popularity. However, it has rarely been considered for use as a household treatment system.

Although the effluent quality from a trickling filter is usually not as good as that from an activated sludge process, the trickling filter is a very attractive treatment method. Aerobic microorganisms are attached to the filter media surfaces and not in the suspended state as in activated sludge. This eliminates the need for mixing, a highly efficient final clarifier, and sludge return facilities. It also makes the system less susceptible to upset. Treatment efficiency drops during hydraulic surges but the biological solids are less likely to be washed out of the system and therefore, recovery is more immediate. Recovery from toxic shock loads is also rapid since only the outermost organisms in the slime are usually affected. These advantages would seem to make the system ideal for single household use.

The disadvantages, however, rule out the use of conventional trickling filters for private systems. The large head requirement of the system dictates deep excavation or pumping above ground structures. These are often undesirable or impossible. Temperature effects on treatment are also great and maintaining temperatures within the filter above those necessary for efficient treatment would be costly.

Three manufacturers, however, do market systems that closely resemble trickling filters. All have modified the process to reduce the headlosses. One manufacturer offers a horizontal folded filter with forced ventilation while another uses a folded aeration tray following a septic tank. The third drives the filter media through the wastewater on a chain-and-bucket principal. These modifications allow below ground construction which provides insulation to reduce temperature effects.

Rotating biological disks--Rotating biological disk units are a series of closely spaced disks mounted on a horizontal shaft such that they can be slowly rotated through the wastewater while partially submerged. The disks provide a large surface area for aerobic organisms to attach themselves and grow as the disks are rotated. The constant rotation brings the organisms into contact with both the wastewater and the atmosphere. When the growth becomes too thick the biological solids are sloughed by the shearing action created by the rotation.

Unlike the other processes discussed, very good control over the degree of treatment is possible by placing several disk modules in series. As the wastewater moves through the system, waste fractions are removed or partially degraded to change the wastewater characteristics. The organisms which can most efficiently utilize the substrate reaching them become dominant. If constant flow is maintained through the use of a holding tank and feed mechanism, a succession of specialized organisms can result providing very effective and rapid treatment.

All the rotating disk units identified on the market today provide a three-step process including pretreatment, biological stabilization on the disk, and final clarification. Usually a septic tank provides the pretreatment of the raw wastewater, but a first stage disk module can also be used to create turbulence to break up the solids. In the latter case, a perforated wall separates the first stage from the second stage. Greases and other floatables also enter the second chamber.

The rotating disks provide secondary treatment. Treatment efficiency depends on the number of stages, the hydraulic loading ($L/m^2/d$), residence time, ratio of liquid volume to disk surface area in each stage, rotational speed, and temperature (Autotrol Inc., 1972). Of these, only rotational speed can be manipulated at the site. A wide variation of the other variables is found among the units on the market and little data are available to allow comparisons of the relative importance of each parameter in treatment efficiency.

Final clarification occurs in settling or upflow sedimentation chambers. Gravity sludge return is generally employed to recycle the settled solids for further decomposition. Return can be made either to the primary or secondary treatment chamber.

The rotating disk process is relatively stable and efficient, and requires little power or maintenance. The solids that are produced and sloughed are generally dense and settle rapidly as in a trickling filter. Theoretically, this system appears well suited to individual wastewater treatment, but operational experience is required.

Submerged biological media system--Submerged biological media systems are similar to rotating disk units but rather than rotating the attached biological growths through the wastewater, the wastewater is continuously aerated and circulated through a submerged system of trays, tubes, or rocks. Aeration and circulation may be provided by an air lift pump. In a system developed by Klock (1973) several cells are put together to achieve a high degree of treatment and stabilize the process. Because of the long solids retention time, sludge production is kept at a minimum. Another aerobic fixed media system employs plastic rings fixed in a 0.58 m^3 (20 cu ft) chamber (St. Louis Ship - Ecodyne, 1974). Mechanical aeration within this chamber circulates wastewater through the media. The fixed bed chamber is preceded by a 3.7 m^3 (130 cu ft) tank contiguous with the fixed bed unit. This system, developed for shipboard applications requires no gravity biological separation step and effluent is discharged directly from the fixed media chamber.

Physical-Chemical Processes

In recent years, physical-chemical processes used to treat municipal and industrial wastewaters have been given considerable attention because of increasing demands for higher quality effluents. Periodic upsets which occur in biological systems due to hydraulic surges, sudden changes in wastewater characteristics, temperature effects, and other unpredictable phenomenon make it very difficult to maintain a high quality effluent. The type of physical or chemical process used depends upon the waste fraction to be removed. Table A-102 presents a list of selected processes that potentially would be available for low-flow waste treatment application.

TABLE A-102. SELECTED PHYSICAL-CHEMICAL PROCESSES AND THE WASTE FRACTIONS REMOVED

Process	Waste Fraction Removed	
	Primary	Secondary
Coagulation/ Precipitation	Colloidal suspended solids, and phosphorus	Some COD, BOD & $PO_4^{\equiv}$ bacteria and viruses
Filtration	Suspended solids	Suspended BOD, COD, $PO_4^{\equiv}$ Possible nitrification
Carbon Adsorption	Dissolved COD, BOD	Residual suspended solids
Ion Exchange	Phosphorus, NH_3, & total dissolved solids	Specific ions depend upon resins selected

Because many of these processes are able to treat highly intermittent and varying strength waste flows without upset, they should be well suited for individual home use if the need for operation and maintenance is sufficiently low. Several manufacturers market such units. Most of the units use the physical-chemical processes as tertiary treatment steps following biological treatment units but a few of the units do provide complete physical-chemical treatment.

Chemical Coagulation and Precipitation--

Coagulation is a process in which chemicals are mixed with the wastewater to form rapid settling solids from the colloidal and suspended particles that cannot be removed through simple sedimentation because of their slow settling velocities. With the proper chemicals, chemical precipitation from solution of phosphorus and other compounds is also realized. Sedimentation follows the chemical addition and flocculation to remove the resulting flocculent solids.

In effective chemical processing of wastes, sufficient chemicals must be added and mixed thoroughly with the wastewater at the proper time. The system used to achieve this must be highly reliable, with little need for supervision and maintenance. To provide the proper dose of chemicals, the wastewater volume to be treated must be known. This can be done in two ways. The first is to use the batch treatment process (Environment/One Corp., 1972). This requires more mechanical equipment but provides excellent treatment. One method involves the accumulation of the wastewater with aeration for mixing and aerobic processing until the tank is filled to a preset volume. Raw waste is temporarily diverted into a holding tank while sufficient chemical is added and mixed. Chemical addition is accomplished by a mechanical pump and mixing is provided by aeration. After mixing, aeration is stopped and the tank acts as a sedimentation chamber.

In continuous flow operations, other means of flow measurement must be used. One such method developed by another manufacturer also employs the batch principal, but on a smaller scale (Anticimexbolagen, 1971). After receiving biological treatment the wastewater flows into a dipper which, when filled empties into a mixing cone before it is discharged to a sedimentation tank. Each time the dipper tips, a preset amount of chemical is added. The mixing cone is designed to provide sufficient turbulence to mix the chemical and wastewater. Although nearly 30 cm of head is lost through this system, it has the advantage of being completely driven hydraulically rather than by mechanical means.

The selection of chemicals to be used involves many considerations. For individual home units, iron salts are very corrosive chemicals, requiring special materials for handling. Lime, though inexpensive and easy to handle, requires an additional neutralization step prior to discharge. Alum is most often used in household units.

Filtration--

Since most individual home filtration systems can be classified as intermittent sand filters this discussion will be limited to this form of wastewater filtration. Intermittent sand filtration may be defined as the intermittent application of raw or pretreated wastewater to an artificial or natural bed of sand which is underdrained to collect and discharge the filter effluent. Many intermittent sand filters are used throughout the United States to treat and dispose of wastewater from motels, restaurants, trailer parks, service stations and individual homes not served by a municipal wastewater system. Sand filters are capable of purifying wastewater to a considerable degree through mechanisms of straining, adsorption and biochemical oxidation. The size of intermittent sand filters range from ten to several hundred m^2. Construction is normally subsurface or covered in northern climates where freezing is a problem while open surface filters are common in warmer climates.

The degree of purification attained by an intermittent sand filter is dependent upon: (1) the type and biodegradability of wastewater applied to the sand filter; (2) the environmental conditions within the sand filter; and (3) the design characteristics of the sand filter. Generally, it has been established that by increasing hydraulic loading rates to sand filters as the quality of the applied wastewater increases, comparable sand filter effluent qualities and length of time between filter clogging will be obtained.

Reaeration and temperature are two of the most important environmental conditions that affect the degree of wastewater purification through a sand filter. Availability of oxygen within the sand pores allows for the aerobic decomposition of the wastewater. Temperature directly affects the rate of microbial growth, chemical reactions, adsorption mechanisms and other factors that contribute to the purification of wastewater within the sand media.

Proper selection of process design variables also aids in the purification of wastewater by intermittent sand filters. A brief discussion of the influence of these design variables on process performance is presented below.

Sand media--The successful use of sand as a filtering media is dependent upon the proper choice of size, shape and uniformity of sand grains. Hazen (1892) developed and defined two descriptive parameters to properly characterize a given filter sand. These parameters are the effective size and the uniformity coefficient. Both are obtained from a mechanical sieve analysis of a representative sample of sand.

The effective size of the sand affects the quantity of wastewater that may be filtered, the rate of filtration, the penetration depth of particulate matter and the quality of the filter effluent. Sand that is too coarse lowers the retention time of the applied wastewater in the filter to a point where adequate biological decomposition is not attained. Too fine a sand limits the quantity of wastewater that may be successfully filtered, due to the low hydraulic capacity and the capillary saturation characteristic of fine sands. Metcalf and Eddy (1935) and Boyce (1927) recommend that not more than 1% of the sand should be finer than 0.13 mm. Many suggested values for the effective size and uniformity coefficient exist in the literature. Table A-103 presents some of these recommended values. Most of these recommendations are similar.

An excellent filter media consists almost entirely of siliceous sand. Total organic matter in the sand should be less than 1% and total acid

TABLE A-103. RECOMMENDED SAND MEDIA CHARACTERISTICS FOR TREATED SEPTIC SEWAGE

	Effective Size (mm)	Uniformity Coefficient
Manual of Septic Tank Practice (1967)	0.25 to 0.6	Less than 4.0
Ten States Standards (1960)	0.4 to 1.0	Less than 3.5
ASCE Committee on Filtering Materials (1937)	0.2 to 0.5	Less than 5.0
J. A. Salvato (1955)	0.25 to 0.5	Less than 4.0

soluble matter should not exceed 3%. Any clay, loam, limestone or other organic material may increase the initial adsorption capacity of the sand but may lead to a serious clogging condition as the filter ages.

Shapes of individual sand grains vary from round to angular configurations. Purification of wastewater infiltrating through sand is dependent upon the adsorption and oxidation of organic matter in the wastewater. To a limiting extent this is dependent upon the shape of the grain size; however, it is more dependent upon the size distribution of the sand grains which is characterized by the uniformity coefficient.

The arrangement or placement of different sizes of sand grains throughout the filter bed is also an important design consideration. Basically, there are four major arrangements of different sand sizes:

1. a homogeneous bed of one effective size sand;
2. a non-homogeneous bed of pit run sand;
3. a bed having coarse sand layers above fine sand layers; and
4. a bed having fine sand layers above coarse sand layers.

A homogeneous bed of one effective size sand does not occur often in practice. Numerous fine and coarse stratified layers of sand occur throughout the bed due to construction practice, thus making it non-homogeneous. In a bed having fine sand layers placed above coarse sand layers, the downward attraction of wastewater is not as great due to the lower amount of cohesion of the water in the larger pores (Clark and Gage, 1909).

The media arrangement of a coarse sand over a fine sand appears to be the most favorable wastewater treatment process for at least two reasons: (1) each coarse layer of sand is underdrained by a finer sand thus increasing the downward pull of water due to the constantly increasing cohesion of the sand layers, and (2) each succeeding finer layer of sand will strain and mechanically remove finer particulate matter from the wastewater (Clark and Gage, 1909). However, it may be difficult to operate such a filter due to internal clogging of the filter.

A properly chosen sand media may be an excellent purifier of wastewater by itself, but its combination with other filter media during construction may seriously degrade its performance. Therefore, proper care must be taken in the construction of the filter bed.

Depths of intermittent sand filters were initially designed to be 120 to 305 cm (4 to 10 ft), however, it was soon realized at the Lawrence Experimental Station (Clark and Gage, 1909) that most of the purification of wastewater occurred in the top 23 to 30 cm (9-12 in) of the bed. Additional bed depth did help to improve the wastewater purification, but not to any significant degree. Most sand depths used today range from 60 to 100 cm (24 to 42 in). The use of shallow filter beds helps keep the cost of installation low. Deeper sand beds tend to produce a more consistent effluent quality and permit more sand cleaning operations before media replacement is required.

Loading rates and patterns--The hydraulic loading rate may be defined as the volume of liquid applied to the surface area of the sand filter over a designated length of time. Values of recommended loading rates for intermittent sand filtration vary throughout the literature depending upon the effective sand size and the quality of influent wastewater. Figure A-34 shows the numerous hydraulic loading rates versus effective sand size employed for previous intermittent sand filter studies.

Expressions of organic loading rates are not often found in the literature. However, it was realized by early investigators that the performance of intermittent sand filters was dependent upon the accumulation of stable organic material in the filter bed (Clark and Gage, 1909; Schwartz, et al., 1967). To account for this, suggested hydraulic loading rates are often given for a particular type of wastewater. Low loading rates are suggested for raw wastewater with increasing loading rates suggested for primary septic tank and secondary effluents.

Dosing techniques refer to methods of application of wastewater to the sand filter. This is somewhat dependent upon whether the sand surface is exposed or buried. Dosing methods that have been used include ridge and furrow application, drain tile distribution, surface flooding, and a spray distribution method. Early sand filters for municipal wastewater were surface units and normally employed ridge-furrow application or spray distributing methods. Intermittent sand filters in use today for motels, restaurants, trailer parks, and service stations are normally built below the soil surface and make use of tile distribution. Installations where freezing is not a problem are now being constructed with the surface of the filter exposed and distribution by flooding of the sand surface. From the Cincinnati study (Schwartz, et al., 1967), surface flooding was found to be a simple distribution method that produced high COD and coliform removals along with good nitrification.

The frequency of dosing intermittent sand filters is open to considerable design judgment. Most of the earlier studies used one dose per day. The Florida studies investigated multiple dosings and concluded that the BOD removal efficiency of sands with effective sizes greater than 0.45 mm is appreciably increased when the frequency of loading is increased beyond twice per day (Emerson, 1945). This multiple dosing concept is being successfully used in recirculating sand filter systems in Illinois (Hines and Favreau, 1974). These installations use a sand media with effective size 0.6 to 1.0 mm and a dosing frequency of 1 per 30 minutes.

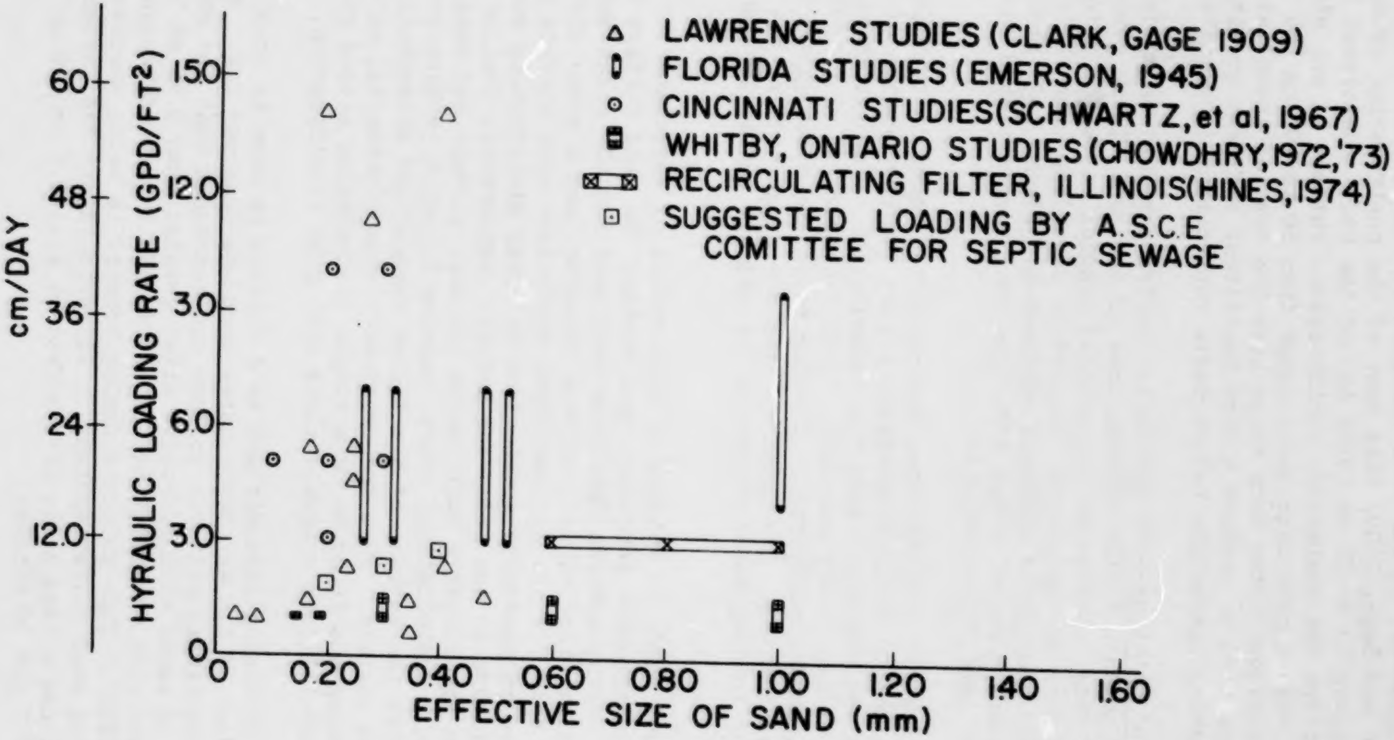


Figure A-34. Summary of previous intermittent sand filter studies.

Performance characteristics--A critical summary of the literature examining the successful performance of intermittent sand filters is shown graphically in Figures A-35 through A-37 (Sauer, 1975). Hydraulic loading rate is plotted versus effective size of sand for septic tank, primary settled and secondary treated effluents. Percent BOD_5 reduction and the length of time between which maintenance is required to sustain continuous operation are the dependent variables. Only sand bed depths greater than 76 cm (30 in) were considered when plotting the graphs. Dosing frequencies of 1 per day, with the exception of the Florida studies, were used predominantly in the literature and, therefore, are used in the graphs. Uniformity coefficients of the sand considered were less than 5.0 thus meeting acceptable media standards.

Due to the many different operating conditions for each intermittent sand filter study, it is important to emphasize that the graphs only represent trends demonstrated from previous studies.

Summary of recommended design from literature--The following statements summarize the information found in the literature:

1. Wastewater passed through sands with effective size less than 0.20 mm is highly purified; however, sands of this size have low hydraulic capacities and, therefore, require frequent maintenance at loading rates greater than 6.1 cm/day (1.5 gpd/ft^2).

2. A major portion of the literature studies were conducted with sands with effective sizes ranging from 0.20 to 0.50 mm and hydraulic loading rates ranging from 4.0 to 24.5 cm/day (1.0 to 6.0 gpd/ft^2).

3. The type and frequency of maintenance required to restore a clogged sand filter bed to successful and continuous operation is not well established in the literature. The required maintenance frequency recorded on Figures A-35 through A-37 is based upon trends of sand filter failures in the literature.

4. Only the Whitby, Ontario intermittent sand filter studies were operated under flow conditions typical of individual households. These subsurface sand filters were operated at 4.1-6.1 cm/day (1.0-1.5 gpd/ft^2).

5. Intermittent sand filters having sand media with effective size greater than 0.20 mm and loaded at less than 6.1 cm/day (1.5 gpd/ft^2) with septic tank, primary settled or secondary treated wastewater require no maintenance for at least 18 months.

6. A majority of the literature studies experienced a minimum BOD reduction of 80% through the sand filter for both septic tank and settled wastewater influents.

7. The graph representing primary settled wastewater was generated almost entirely from data from the Florida study. The majority of these experiments were conducted using multiple dosing techniques. Observation of the graph, Figure A-37 shows that percent BOD reduction is less dependent on hydraulic loading rate. From this, one might conclude that

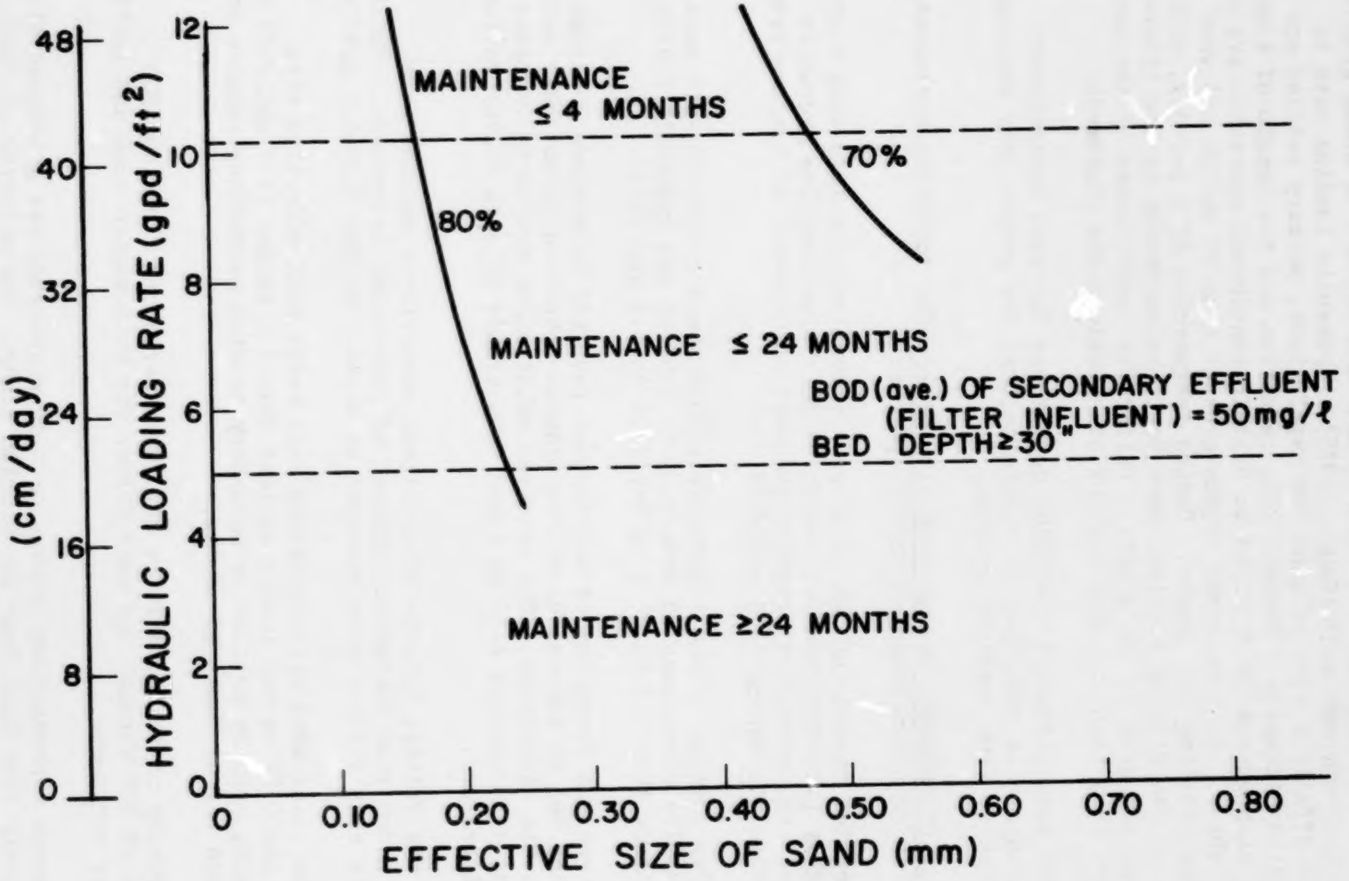


Figure A-35. Trends of percent BOD reduction and required maintenance of literature sand filters treating secondary treated wastewater (Sauer, 1975).

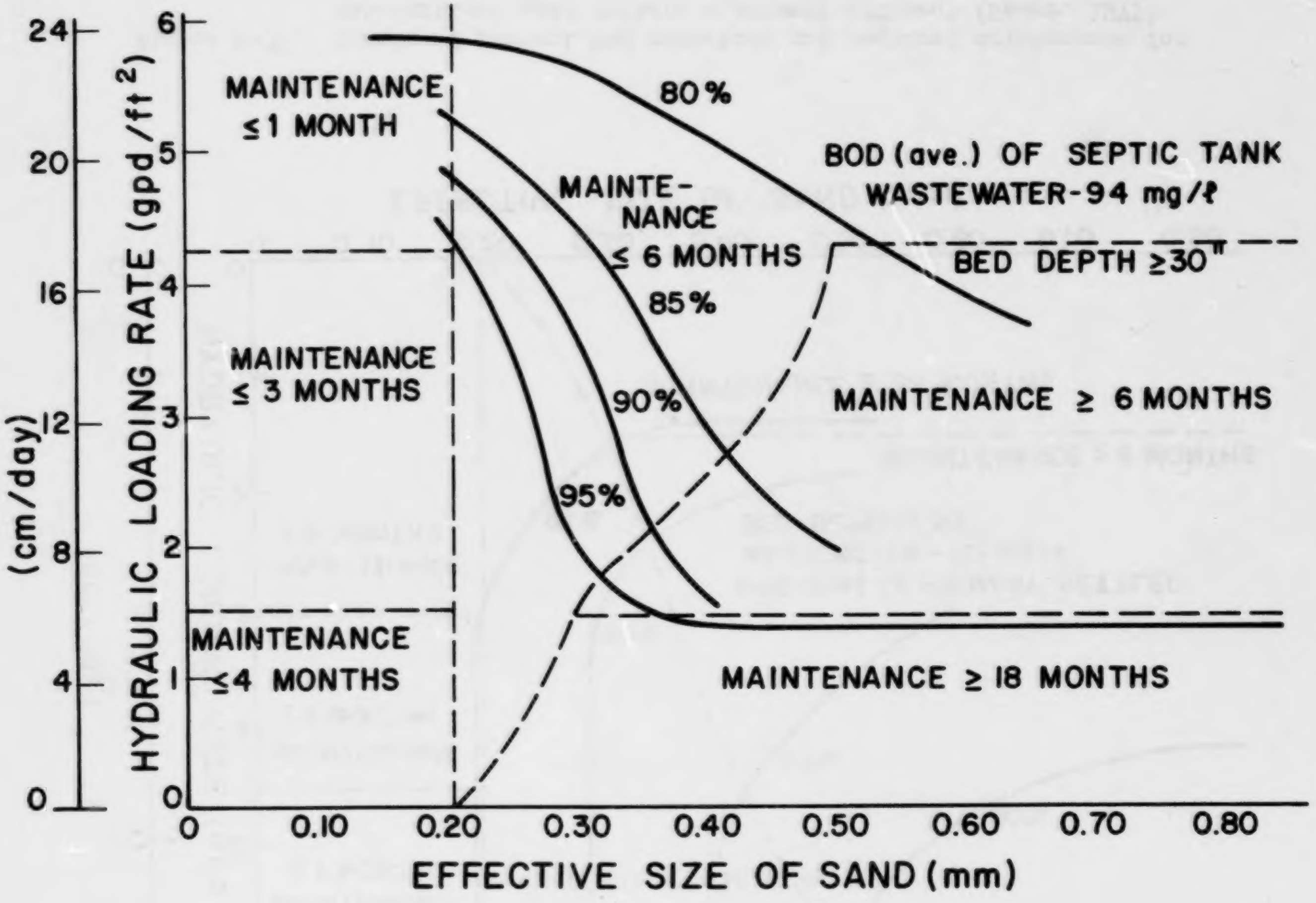


Figure A-36. Trends of percent BOD reduction and required maintenance of literature sand filters treating septic tank wastewater (Sauer, 1975).

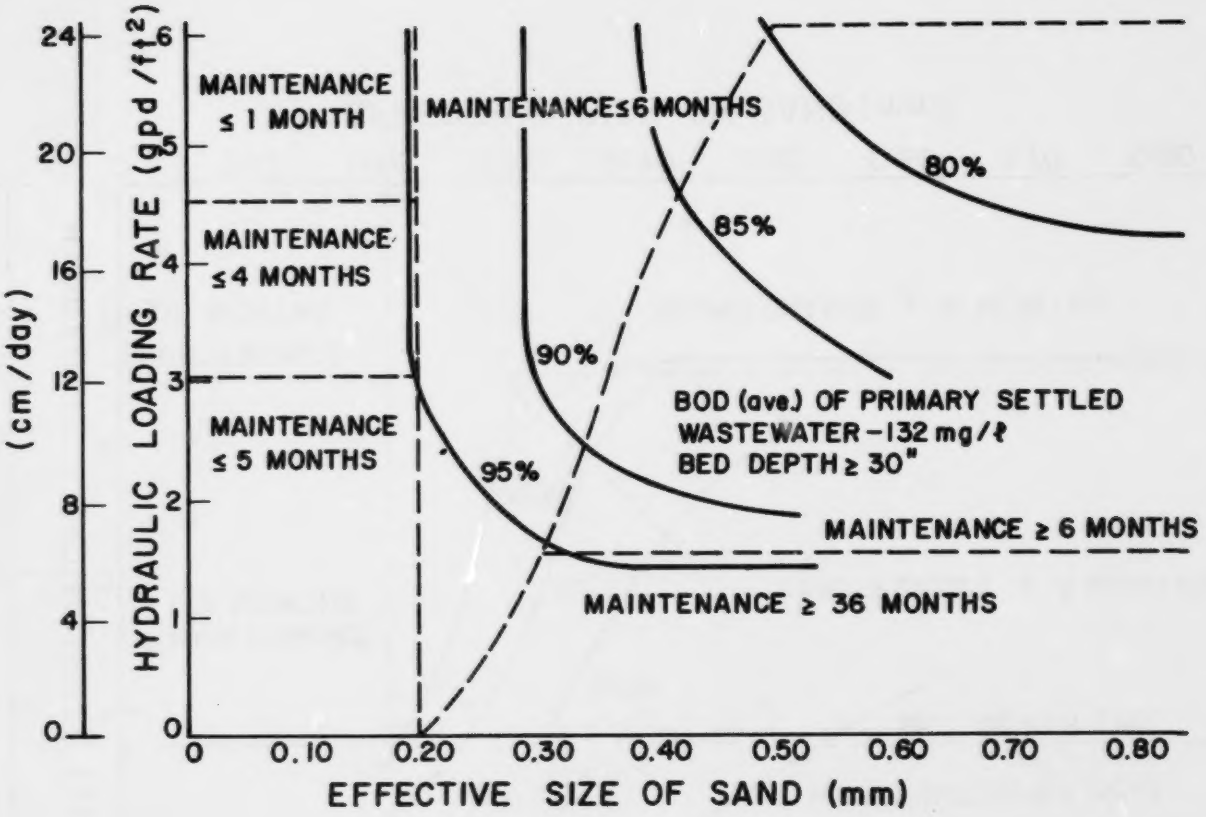


Figure A-37. Trends of percent BOD reduction and required maintenance for intermittent sand filters - primary effluent (Sauer, 1975).

multiple dosing techniques reduces the dependency of BOD_5 reduction on hydraulic loading rate. This conclusion is substantiated in part by the graph on septic tank treated wastewater. Sand filters represented by this graph were dosed once per day, and it is apparent that percent BOD_5 reduction is not independent of hydraulic loading rate.

8. Intermittent sand filtration of secondary treated wastewater is not well established in the literature as shown in Figure A-36.

Ion Exchange--

Ion exchange is a widely accepted method for the removal of inorganic anions and cations from waters. Its use on an individual household basis is well documented through the use of millions of cation exchangers for household water softening and iron removal. In wastewater treatment, its primary use has been in the removal of ammonia, and it also has been used for the removal of heavy metal cations, anions of phosphate, nitrate, anionic metal ion complexes and some organic molecules. A serious problem with the use of ion exchange for treatment of wastes with substantial organic content is resin binding caused by the organic matter. Prefiltering of waste or use of scavenger resins has partially solved this problem (Applebaum, 1968). Both synthetic and natural resins are employed in ion exchange applications. Reviews of applications of ion exchange processes to wastewater can be found in Eliassen, et al. (1965), Eliassen and Bennett (1967), Mercer, et al. (1970), and Weber (1972).

In wastewater treatment, the removal of nitrogen as ammonia or possibly, nitrate, employing ion exchange techniques has been successfully applied (Battelle Northwest, 1969). The ion exchange resin normally employed is a natural zeolite, Clinoptilolite. This naturally occurring resin is found in the bentonite deposits of the Western U.S. It is a hydrated alumino-silicate material of the general composition $(MN_2)\ O \cdot Al_2O \cdot mSiO_2 \cdot nH_2O$ where M and N are the alkali metal and alkaline earth counter ions, respectively. Most U.S. deposits are of the sodium form. The order of its selectivity is $Cs^+ > Rb^+ > K^+ > NH_4^+ > Ba^{++} > Sr^{++} > Na^+ > Ca^{++} > Fe^{+3} > Al^{+3}$ Mg > Li. A detailed review of studies with Hector Clinoptilolite may be found in the literature; Battelle Northwest (1971), Mercer, et al. (1970), Koon and Kaufman (1971), McLaren and Farquhar (1973), and Jorgensen, et al (1976).

ANAEROBIC/AEROBIC UNIT STUDIES

The objective of this phase of the Small Scale Waste Management Project was to evaluate the present state-of-the-art in household wastewater treatment. Included in this evaluation were conventional septic tanks, aerobic units, and intermittent sand filters.

It is felt that the performance of any treatment system should be evaluated according to the following criteria: (1) the average effluent quality produced, (2) the variability in effluent quality, (3) the operating and maintenance requirements, and (4) the total annual cost. If a system is to be designed to produce a particular effluent quality, it is necessary to know the capabilities and limitations of each component (e.g. septic tank, sand filter, etc.) of that system according to these four criteria. A unit may produce an effluent of high average quality, but if it is highly variable and unpredictable, then the subsequent treatment and disposal methods must be chosen and designed to handle these fluctuations. This leads to a more conservative design and a more expensive system. A system with few mechanical parts is also desirable to simplify maintenance and to prevent frequent breakdown. This is important to improve reliability. The total annual costs are the final criteria, including capital, installation, operation, and maintenance costs. It is desirable to keep the cost at a minimum while still achieving the necessary effluent quality.

Experimental Methods

Full-sized treatment units were installed at several locations throughout the State of Wisconsin under both laboratory and field conditions (Table A-104). A total of 11 septic tanks, 11 aerobic units (9 different manufacturers), 1 chemical unit and 4 sand filter units were evaluated. Unit installations began in the Fall of 1971 with the last one being made during the Summer of 1975 (field site J).

Laboratory Investigations--

The laboratory investigations were conducted at two sites (site M from January, 1973 to August, 1973 and site N from January, 1975 to March, 1976). The primary objectives of the laboratory studies were to compare selected treatment processes under controlled conditions, to identify critical design features of these selected units, and to determine operation and maintenance requirements.

Description of Installations--The unique feature of the laboratory studies was the use of a specially designed wastewater simulator (Hutzler, 1974) (see Attachment D for a detailed description of laboratory site N). This simulation removed much of the home to home influent wastewater variation so that variations seen in effluent quality between units could be attributed to variations within each process. Major household water-use events were simulated through specially designed equipment and the resulting wastewaters were directed to the various treatment units according to schedules as outlined in Attachment D. The units were fed wastewaters generated to simulate clotheswashing, dishwashing, kitchen wastes, garbage grinding, bathing, showering, and toilet use. The flow rates and volumes of the various events at each laboratory site are tabulated in Table A-105. Each process received the same schedule of events; however, the starting times were staggered to avoid duplication of the equipment. The schedule and strength of events were selected on the basis of previous studies on the characterization of wastewater (Ligman, 1972; Witt, 1974a; Siegrist, 1975). The monthly averages of the influent wastewater characteristics were determined by measuring the characteristics of the feed materials used and by keeping accurate records of their utilization. The average values for sites M and N are listed in Table A-106.

Typical treatment units were selected and installed at the two laboratory sites (Table A-104). Six treatment units were installed in an underground sewage lift station (site M) and an additional 5 units were later installed

TABLE A-104. TABULATION OF TREATMENT UNITS

Series/Site	Unit Type	Capacity	Comments
Field Site			
A	Septic Tank	3.8 m^3	-
B	Septic Tank	3.8 m^3	-
C	Septic Tank	3.5 m^3	-
C	Extended Aeration	2.8 m^3/d	Receives septic tank effluent
D	Septic Tank	4.5 m^3	-
E	Septic Tank	3.8/1.9 m^3	Two tanks in series followed by sand filter
F	Septic Tank	3.8 m^3	-
G	Batch Aeration*	1.5 m^3/d	-
H	Extended Aeration*	2.3 m^3/d	Followed by sand filter (preceded by septic tank during second phase of test)
I	Rotating Disks*	1.5 m^3/d	-
J	Septic Tank	3.8/3.8 m^3	Two tanks in series followed by sand filter
J	Extended Aeration	1.5 m^3/d	Receives septic tank effluent and is followed by sand filter
Laboratory Site			
M	Septic Tank	2.3 m^3	-
M	Septic Tank	1.0 m^3	Three compartments
M	Batch Aeration*	1.5 m^3/d	Same as unit listed under N
M	Extended Aeration*	2.3 m^3/d	-
M	Chemical Addition	1.1 m^3	Received effluent from 1.0 m^3 septic tank
M	Rotating Disk	1.1 m^3/d	Received effluent from 2.3 m^3 septic tank; manufacturer provided disks module only
N	Septic Tank	4.0 m^3	-
N	Septic Tank	2.0 m^3	-
N	Batch Aeration*	1.5 m^3/d	Received 2.0 m^3 septic tank effluent
N	Extended Aeration	1.8 m^3/d	-
N	Rotating Disks*	1.5 m^3/d	-
N	Submerged Media	1.5 m^3/d	-

* Similar to unit tested by NSF.

TABLE A-105. INFLUENT CHARACTERISTICS

	Laboratory Sites					
	Lab Site M			Lab Site N		
Event	Flow (L/min)	Volume of Event (L)	Daily Volume (L)	Flow (L/min)	Volume of Event (L)	Daily Volume (L)
T -Toilet	57	11	165	57	15	195
B -Bath	75	95	95	40	87	87
S -Shower	9.5	95	95	11	87	174
L -Laundry	60	150	150	60	185	185
DW-Dishwashing	7.5	55	110	7.5	57	57
K -Kitchen Waste	11	8	15	11	14	42
G -Garbage Grinding	11	15	30	-	-	-
Daily Flow	-	-	660 (175 gpd)	-	-	740 (195 gpd)

Volume in gallons = Volume in liters x 0.264.

at site N (the batch aeration unit was moved from one site to the other). The units at site M were: (1) a 2.3 cubic meter (610 gal), single-chambered septic tank; (2) a 1.0 cubic meter (260 gal) multi-chambered septic tank; (3) a 1.1 cubic meter (290 gallon), chemical addition and precipitation unit; (4) a 4.5 cubic meter (1200 gal), extended aeration unit; (5) a 2.4 cubic meter (660 gal), batch aeration unit; and (6) a rotating disks module (23 m^2 surface area) with a 0.59 cubic meter (155 gal) clarifier. Space constraints at the lift station precluded the installation of full-sized septic tanks. The rotating disk unit consisted of a 0.48 m diameter module provided by the manufacturer and a rectangular clarifier constructed by project personnel. The remaining units were installed according to manufacturer specifications. Figure A-38 presents a diagram of laboratory site M.

The treatment units installed at site N included: (1) a 4.0 cubic meter (1000 gal) single-chambered septic tank; (2) a 2.0 cubic meter (500 gal) single-chambered septic tank; (3) a 2.4 cubic meter (660 gal) batch aeration unit (from site M); (4) a 3.0 cubic meter (750 gal) extended aeration unit, (5) a 4.0 cubic meter (1000 gal) submerged media unit; and (6) a 2.3 cubic meters per day (6000 gpd) rotating disks unit (44 m^2 surface area) installed with a 4.0 cubic meter (1000 gal) septic tank. All the site N treatment units were obtained from individual manufacturers; however, the rotating disks and the submerged media units were considered as prototypes. The simulation program was modified somewhat from site M as garbage grinding wastes were eliminated and the waste strengths were altered upward. Wastewater was collected from feeders simulating clotheswashing, dishwashing, kitchen sink wastes, bathing and showering, and then directed to the various treatment units via an automated, rotating distributor (Figure A-39). Attachment D describes the laboratory layout in much greater detail. The units were fed a daily flow of 0.74 cubic meters per dav (196 gpd) which had the average characteristics summarized

TABLE A-106. LABORATORY INFLUENT WASTEWATER

Laboratory Site M

Month	BOD_5	COD	TSS
February 1973	165	360	185
March 1973	185	390	180
April 1973	150	315	175
May 1973	160	325	160
June 1973	260	520	320
July 1973	240	485	290
August 1973	200	400	230
AVERAGE	195	400	220

Average Flow = .66 m^3/d (175 gpd).
All values are expressed as mg/L.

Laboratory Site N

Month	BOD_5	COD	TS	TSS	TN	TP
January 1975	200	460	855*	245	50	37
February 1975	155	370	730	220	28	30
March 1975	155	385	830	230	50	37
15th	185	445	935	265	53	44
April 1975	195	435	870	255	53	34
May 1975	185	445	875	250	57	33
June 1975	190	460	920	260	70	38
July 1975	180	430	890	240	58	34
August 1975	190	450	935	255	65	41
September 1975	200	515	1025	310	69	45
October 1975	190	495	980	300	61	43
November 1975	190	470	1015	300	72	47
December 1975	200	510	1010	310	65	46
January 1976	205	500	980	290	61	45
February 1976	215	530	960	315	63	43
March 1976	170	460	1010	295	66	47
WEIGHTED AVERAGE	190	460	922	270	58	41
AVERAGE HOUSEHOLD VALUES	300		950	250	44	25

* Values include 300 mg/L from carriage water.
Average Flow = 0.74 m^3/d (195 gpd).

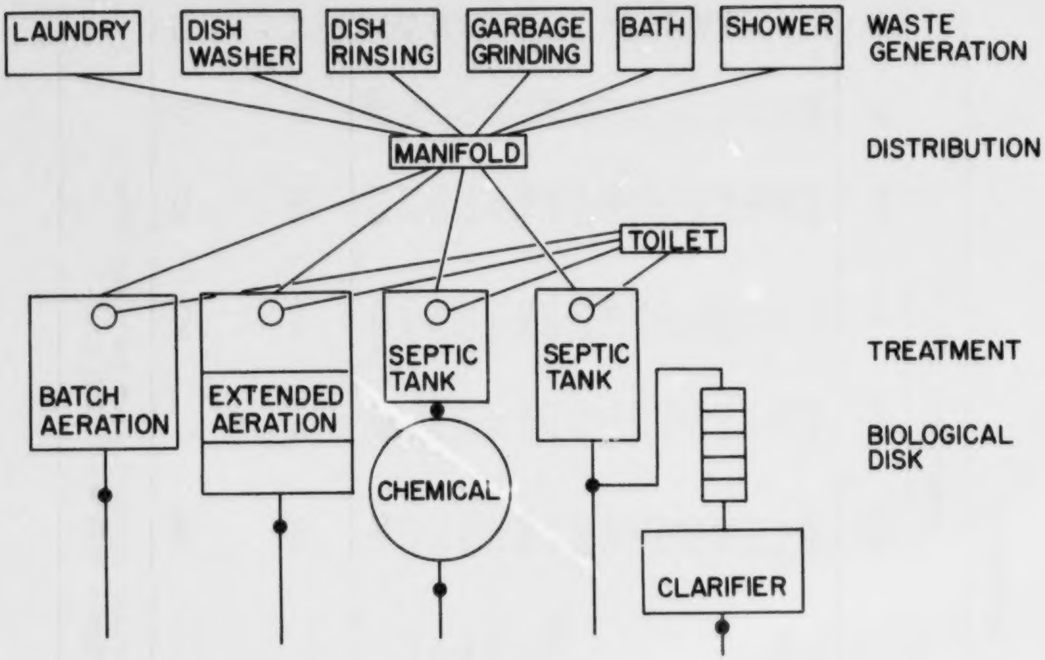

LAUNDRY
DISH WASHER
DISH RINSING
GARBAGE GRINDING
BATH
SHOWER
WASTE GENERATION
MANIFOLD
DISTRIBUTION
TOILET
BATCH AERATION
EXTENDED AERATION
SEPTIC TANK
SEPTIC TANK
TREATMENT
BIOLOGICAL DISK
CHEMICAL
CLARIFIER

Figure A-38. Laboratory flow sheet, lab site M.

in Table A-106. The monthly variations were due either to periodic malfunctions of the feed equipment or to deliberate changes made to increase the waste strength.

Effluents from the 4 cubic meter, septic tank and the extended aeration unit at site N were used to load soil columns and columns representing sand filters. Effluent from the submerged media unit was used to test an ultraviolet irradiation unit.

Sampling--Effluents from all the laboratory units were flow-composited at locations indicated on Figures A-38 and A-39 and collected semi-weekly for analysis. The analyses included: BOD_5 (filtered and unfiltered), COD, solids, nitrogen forms and phosphorus forms and were performed according to Standard Methods (1971). At the time of sample collection, additional samples of the extended aeration and the batch aeration mixed liquors were taken to determine solids content and the sludge volume index.

Miscellaneous--Operation and maintenance requirements were recorded including power consumption, routine maintenance and periodic malfunctions. Necessary repairs were made by project personnel with the assistance of manufacturer representatives.

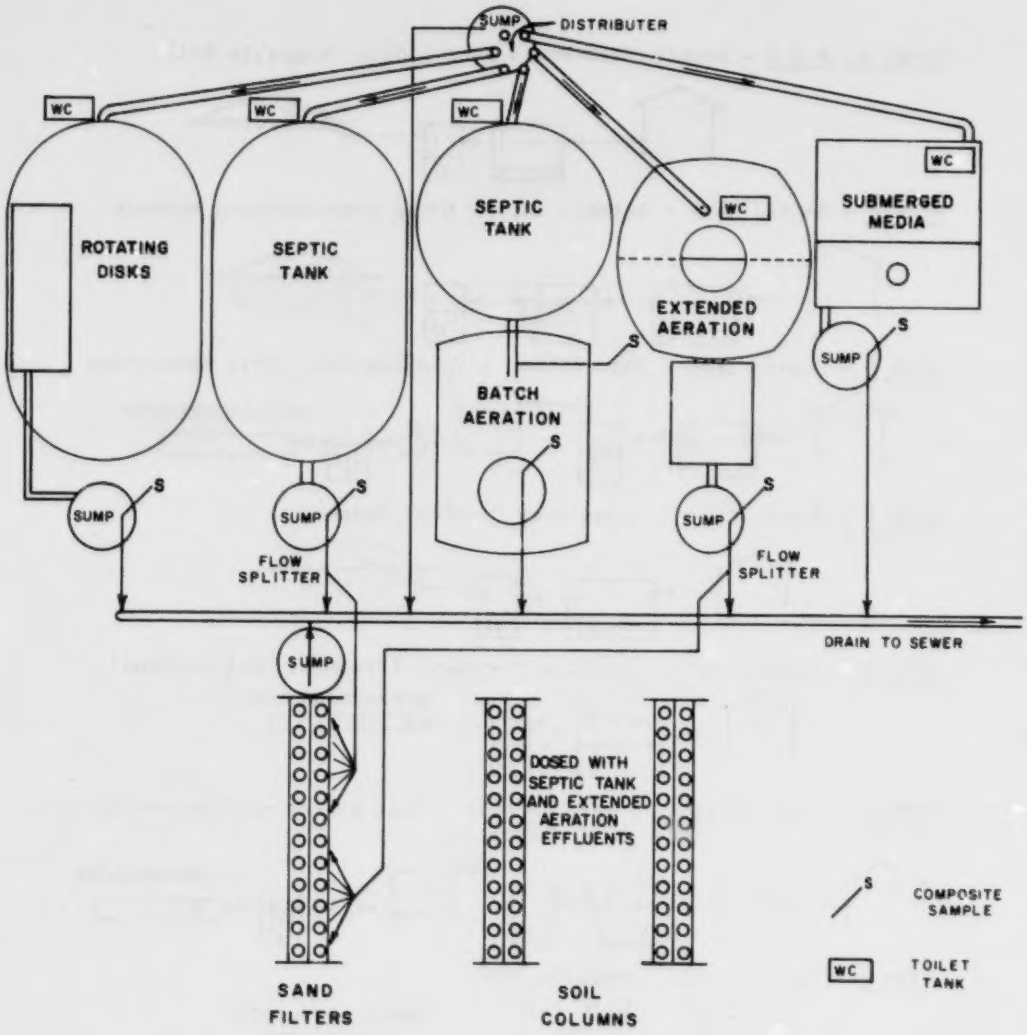

SUMP
DISTRIBUTER
WC
WC
WC
WC
WC
ROTATING DISKS
SEPTIC TANK
SEPTIC TANK
EXTENDED AERATION
SUBMERGED MEDIA
BATCH AERATION
S
SUMP
FLOW SPLITTER
FLOW SPLITTER
DRAIN TO SEWER
DOSED WITH SEPTIC TANK AND EXTENDED AERATION EFFLUENTS
COMPOSITE SAMPLE
TOILET TANK
SAND FILTERS
SOIL COLUMNS

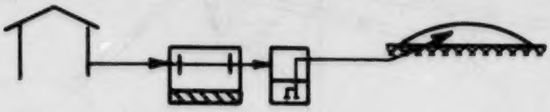
Figure A-39. Laboratory layout, lab site N.

Field Investigations--

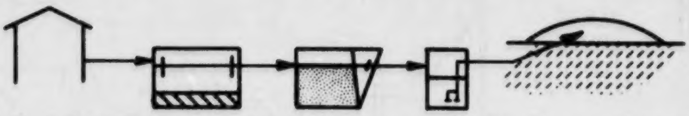
The field investigations began during the Spring of 1972 with data being collected at some installations up to the Spring of 1977. The objectives of the field studies were to observe the installation and operation of treatment units under actual field conditions, to determine the effluent quality of septic tank, aerobic units and sand filters, and to obtain information on operation and maintenance requirements (Otis and Boyle, 1976).

Description of installations--A total of ten installations were made at private homes and on University experimental farms in Wisconsin (Table A-104 and Figure A-40). (See Tables A-13 and A-14 for details on family characteristics for these homes.) The individual installations were made by local

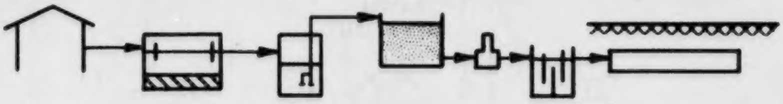
SITES A, B & D - Septic Tank-Mound over Slowly Permeable Soil



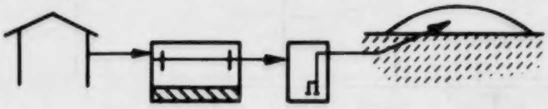
SITE C - Septic Tank - Aerobic Unit - Mound over Creviced Bedrock



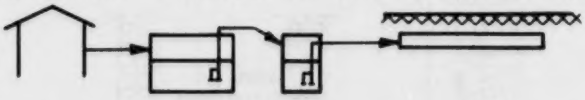
SITE E - Septic Tank - Sand Filter - Chlorination - Soil Absorption



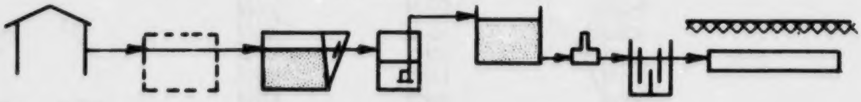
SITE F - Septic Tank - Mound over Creviced Bedrock



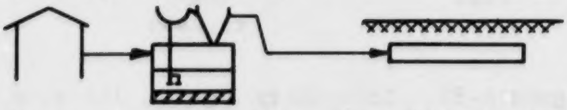
SITE G - Aerobic Unit - Shallow Trenches (Pressure Distribution)



SITE H - (Septic Tank) - Aerobic Unit - Sand Filter - Chlorination - Soil Absorption



SITE I - Aerobic Unit - Shallow Bed



SITE J - Septic Tank - Aerobic Unit - Sand Filters - UV Light - Denitrification - Shallow Beds

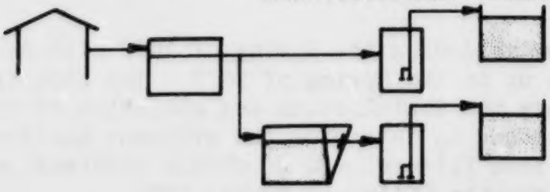


Figure A-40. Schematics of field installations.

contractors with the assistance of project personnel. With the exception of the rotating disks unit, all the treatment units (septic tanks and aerobic units) were typical of designs commercially available at the time of selection.

The sand filters were specially designed to receive application rates of up to 0.4 m^3/m^2d (10 gpd/ft^2). The surface area of the filters was approximately 1.5 square meters (16 ft^2) with sand depths ranging from 0.6 to 0.9 meters (2 to 3 ft). Two of the sand filter units received septic tank effluent while the other two followed aerobic units.

Sampling--Twenty-four hour flow composited samples were collected monthly or semi-monthly by automatic sampling devices from each step in the treatment process. These were analyzed for filtered and unfiltered 5-day biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), solids, total nitrogen (including organic, ammonia and nitrite-nitrate nitrogen), phosphorus, total and fecal coliforms, and fecal streptococcus according to Standard Methods, 13th edition (1971). Temperature and pH were measured at the time of sample collection. Operation and maintenance requirements were also recorded including power consumption and routine maintenance. Necessary repairs of the treatment units were made by local firms, and the costs were recorded. Sand filter operation and maintenance was performed by project personnel. The periods over which data were collected appears in Table A-107.

Data Analysis--

All of the effluent quality data (field and laboratory) were cataloged and analyzed statistically by computer. Tests were made to determine whether the data were distributed either normally or log-normally. In the case of the field data, this determination was made on the basis of the coefficient of skewness while the laboratory data were plotted as histograms and the decision was made by visual observation. The mean, the 95% confidence interval of the mean, and the coefficient of variation were estimated for each parameter based upon the appropriate distribution.

The performance of each unit was measured by the mean and coefficient of variation of each selected parameter. The coefficient of variation is obtained by dividing the standard deviation of the data by the mean (for data distributed log-normally, the coefficient of variation is obtained by dividing the standard deviation of the logarithms of the data by the mean of the logarithms). Smaller coefficients of variation generally indicate greater stability. The 95% confidence intervals of the mean for a selected parameter are used to indicate whether or not a significant difference exists between the effluent quality of the various treatment units. An overlap of the intervals indicates that no significant difference exists at the 95% confidence level.

Treatment Unit Results

Data on the operation and maintenance and on the effluent quality of the various treatment units were collected over a five year period (1972-1976) and are summarized herein. In general, the period of operation for a particular unit was greater than the period of effluent sampling, with start-up periods being eliminated from the statistical analyses. All malfunctions occurring during the period of operation were noted.

TABLE A-107. PERIODS OF DATA COLLECTION AT FIELD SITES

Site	Family* Unit	Water Use Study (Witt, 1974)	Wastewater Characterization (Siegrist, 1975)	Treatment Unit Effluent	Sand Filter Effluent
A	B	Summer 1973, 14 days	-	May 1972- May 1974	-
B	E	Summer 1973, 28 days	-	May 1972- May 1974	-
C	D	Autumn 1973, 42 days	-	July 1972- June 1974	-
D	J	Summer/Winter 1973-1974 68 days	-	Aug. 1972- June 1974	-
E	K	Summer/Winter 1973-1974 62 days	-	Oct. 1972- Dec. 1976	Sept. 1973- Dec. 1976
F	H	Autumn 1973, 24 days	-	Nov. 1972- June 1974	-
G	F	Winter 1974, 28 days	-	Nov. 1972- June 1974	-
H	C	Summer/Winter 1973-1974, 77 days	Spring 1974	Aug. 1973- May 1974	Sept. 1973- May 1974
	C´	-	Summer 1974	June 1974- Dec. 1976	June 1974- Dec. 1976
I	G	Winter 1973-1974, 35 days	Spring/Summer 1974	-	-
J	-	-	-	Mar. 1975- Dec. 1976	June 1976- Dec. 1976
-	I	Winter 1973-1974 28 days	Spring/Summer 1974	-	-

* Designation used in Part 1; See Tables A-13 and A-14.

Laboratory Installations--

Operation--The treatment units installed at the laboratory sites M and N are compared as to their components, normal operation, power consumption, oxygen transfer efficiencies and malfunctions in Tables A-108 and A-109. There was a vast range in complexity of processes from the rather simple operation of the septic tanks to the complex, mechanical operation of the rotating disks unit at site N. The processes are also differentiated in terms of their mechanical components, flow character, primary biological state (aerobic or anaerobic), solids separation and return, and time cycles of operation. Some units included a pretreatment step (extended aeration and chemical addition at site M; batch aeration and rotating disks at site N) which acted to remove gross solids prior to the process treatment.

All the aerobic units were capable of providing sufficient aeration; however, the oxygen transfer efficiencies (kilograms of oxygen transferred per kilowatt-hour of power supplied) varied considerably. The rotating disk unit (site N) had the lowest efficiency while the batch aeration had the highest. Power consumption ranged from 2.4 (rotating disks - site N) to 7.4 kilowatt-hours per day (extended aeration - site N). Some manufacturers have tried to reduce power consumption by either reducing the size of the aerator motor (rotating disks - site N) or by providing an electrical timer for intermittent operation (extended aeration - site M).

Some units were equipped with flow equalization equipment in an effort to reduce effluent variability. The most controlled discharge occurred with the batch aeration unit where the effluent is pumped out at a constant rate after a preset settling period. The flow rate through the rotating disks module (site N) was set at a constant 0.18 cubic meters per day (490 gpd) by a special pump arrangement. With other units, the effluent flow rate was directly affected by the influent flow rate. Several of the laboratory units were monitored as to their effluent flow in response to influent flows. Figure A-41 shows the variation in effluent flow in response to the set schedule of events at laboratory site N. The rotating disks produced the lowest flow variation while the submerged media process had the greatest. More detailed information on these responses is presented in Figures A-42 and A-43. In those tanks not equipped with flow dampening devices (e.g. the septic tank and the submerged media unit), the effluent flow response is a function of two factors, tank surface area and inlet/outlet configuration.

The treatment units varied considerably in the method used for solids separation and solids return. For example, the septic tank is primarily designed as a solids settling and scum floatation process but it is also used for solids decomposition. There was noticeable gas production from the septic tanks but no attempt was made to quantify or determine its composition. This gas production caused a portion of the decomposing solids to float to the surface where they became part of the scum layer. The batch aeration process probably had the most positive method of solids return since aeration and settling occurred in the same chamber. A primary concern with this process is the possibility of influent flow during effluent pump-out; however, since pump-out is normally scheduled for sleeping hours, this is not a likely occurrence.

TABLE A-108. PROCESS COMPARISONS - OPERATION AND MAINTENANCE LABORATORY SITE M

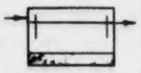
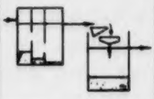
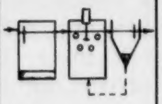
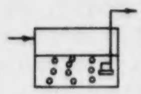
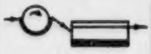
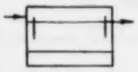
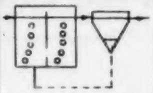
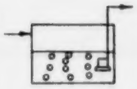
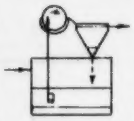
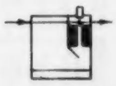
TREATMENT PROCESS	SEPTIC TANK	CHEMICAL ADDITION	EXTENDED AERATION	BATCH AERATION	ROTATING DISKS
Sketch of Process					
Primary Biological State	Anaerobic Decomposition	Anaerobic Decomposition	Aerobic, Suspended Growth	Aerobic, Suspended Growth	Aerobic, Attached Growth
Mechanical Components	None	Tipping Bucket, Float Valve	Aerator Motor, Timer Controlled Alarm	Air Compressor, Submersible Pump, Timer Controlled, Alarm	Gear Motor, Chain Drive
Volume	2.3 m^3	Septic Tank - 1.0 m^3 Clarifier - 1.1 m^3	4.5 m^3 (1.7 - 2.2 - 0.6)	Variable, 1.2 to 2.6 m^3	0.19 m^3 disks; 0.8 m^3 - disks plus Clarifier
Flow Character	Continuous Flow	Continuous, Chemical Addition Metered By Tipping Bucket	Continuous Flow	Batch Discharge	Slightly Equalized, Received Flow From Septic Tank
Solids Separation	Gravity Settling, Scum Floatation	Gravity Settling, Scum Floatation, Chemical Precipitation	Gravity Settling, Scum Floatation, Upflow Clarifier	Aeration and Settling Occur in Same Chamber	Gravity Settling
Time Cycle	None	Chemcial Metered Proportional to Flow	Aerator ON 20 min, OFF 10 min	Aerator ON 20 hr, OFF 4 hr, Pump-Out After 3½ hr of Settling	None
Period of Operation	August 1972-September 1973	October 1972-September 1973	May 1972-September 1973	February 1972-September 1973	December 1972-September 1973
Measured Power Consumption	None	None - Requires About 0.7 kg of $AlSO_4$/d	3 kwh/d (Aerator A-2.8) (Aerator B-3.2)	6.3 kwh/d	3.8 kwh/d
Power Input/Vol.	-	-	.04 kw/m^3	0.17 kw/m^3	0.83 kw/m^3
Oxygen Transfer Efficiency	-	-	Aerator A - 0.13 kg O_2 /kwh Aerator B - 0.27	0.48 to 0.65 kg O_2 /kwh	Not Determined
Required Routine Maintenance	Solids Removal	Solids Removal, Chemcial Replenishment	Solids Removal	Solids Removal	Solids Removal, Parts Lubrication
Malfunctions	None	Float Valve Clogged, Float Valve Failed, Tipper Bucket Unbalanced	Timer Corroded, Shaft Fell Off Aerator Shortly After Installation	Control Panel Problems	Chain Corroded

TABLE A-109. PROCESS COMPARISONS - OPERATION AND MAINTENANCE LABORATORY SITE N

TREATMENT PROCESS	SEPTIC TANK	EXTENDED AERATION	BATCH AERATION	ROTATING DISKS	SUBMERGED MEDIA
Sketch of Process					
Primary Biological State	Anaerobic Decomposition	Aerobic, Suspended Growth	Aerobic, Suspended Growth	Aerobic, Attached Growth	Aerobic, Attached Growth
Mechanical Components	None	Air Compressor, Automatic Valve, Timer Controlled, Alarm	Air Compressor, Submersible Pump, Time Controlled, Alarm	Gear Motor Pump, Dump Valve, Timer Controlled, Alarm	Aerator Motor
Volume	4.0 m^3	Variable 2.7 to 3.1 m^3	Variable 3.2 to 4.6 m^3 includes 2.0 m^3 septic tank	Disk Module- 0.24 Septic Tank - 2.0 to 3.7 m^3	Media - .57 m^3 Total Tank = 3.8
Flow Character	Continuous Flow	Equalized by Air Lift Pump	Batch Discharge	Equalized Constant Flow Thru Disks	Continuous Flow, Recirculation Thru Media
Solids Separation	Gravity Settling, Scum Floatation	Plate Settlers, Air Lift Sludge Return	Aeration and Settling Occur in Same Chamber	Upflow Clarifier, Contents Returned to Septic Tank Daily	Solids Sloughing Off Media Returns to Quiescent Zone by Gravity
Time Cycle	None	Air Lift Sludge Back to Aeration Twice A Day	Aerator ON 20 hr, OFF 4 hr, Pump-OUT After 3½ hr of Settling	Dump Valve Opens Once per Day	None
Period of Operation	September 1974-April 1977	September 1974-April 1976	September 1974-January 1976	September 1974-November 1975	September 1974-April 1976
Measured Power Consumption	None	7.4 kwh/d	6.3 kwh/d	2.4 kwh/d	3.9 kwh/d
Power Input/Vol.	—	0.11 kw/m^3	.17 kw/m^3	.42 kw/m^3	.28 kw/m^3
Oxygen Transfer Efficiency	—	0.11 kg O_2 /kwh	0.48 to 0.65 kg O_2 /kwh	0.061 kg O_2 /kwh	Not Determined
Required Routine Maintenance	Solids Removal	Solids Removal, Parts Lubrication, Unit Cleaning	Solids Removal	Solids Removal, Parts Lubrication, Unit Cleaning	Solids Removal
Malfunctions	None	Valve Failed, Air Line to Air Lift Frequently Clogged, Coupling Between Motor and Blower Wore Out	Pump Failed, Diffusers Clogged, Diffusers Deteriorated	Pump Clogged, Pump Failed, Shaft Broke, Alarm Shorted Out, Disks Module Became Unglued	None

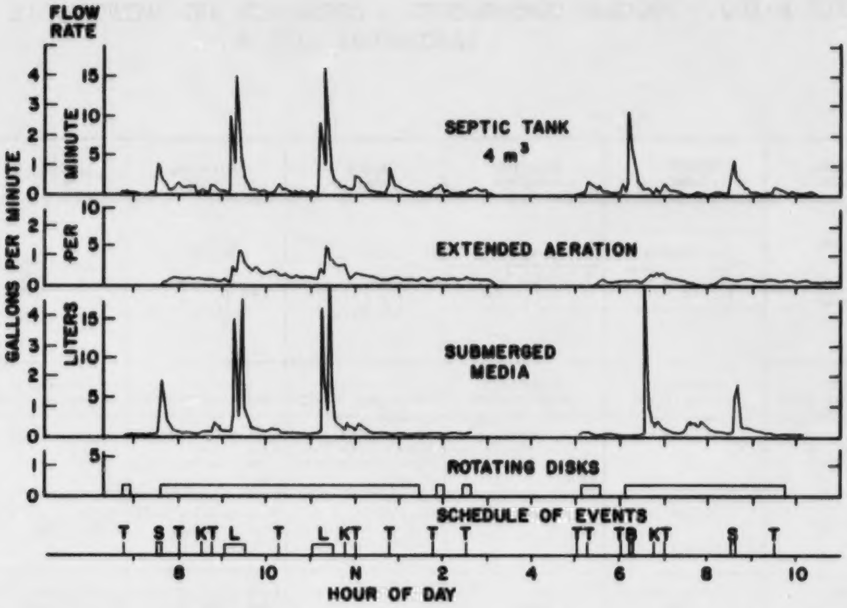


Figure A-41. Flow variations of treatment unit effluents, laboratory site N.

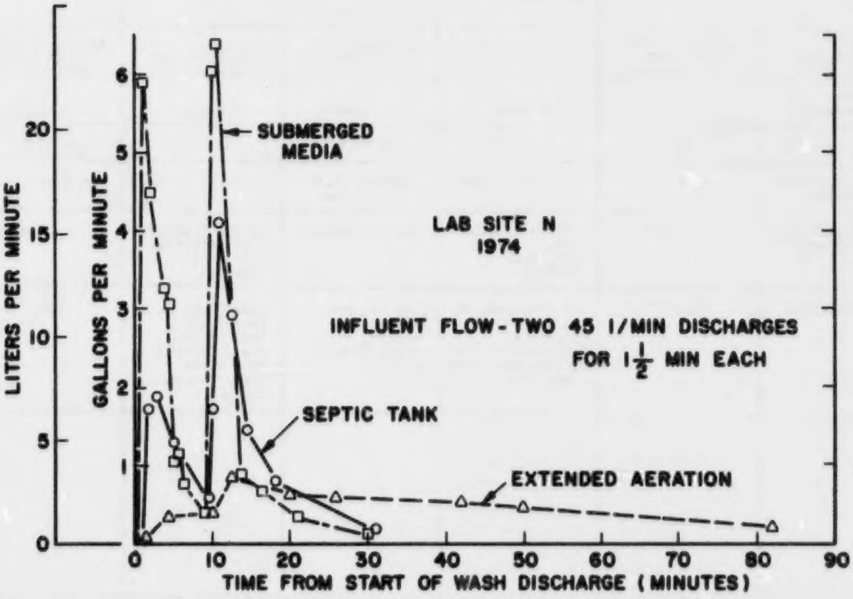
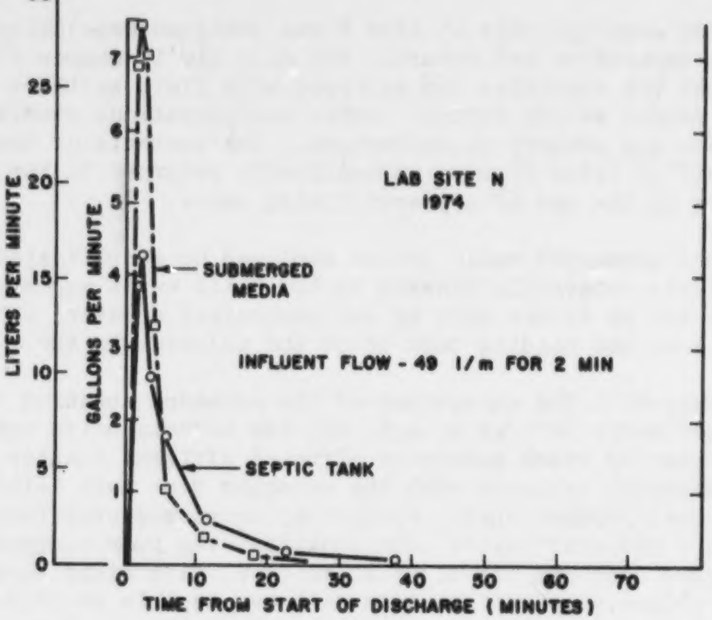
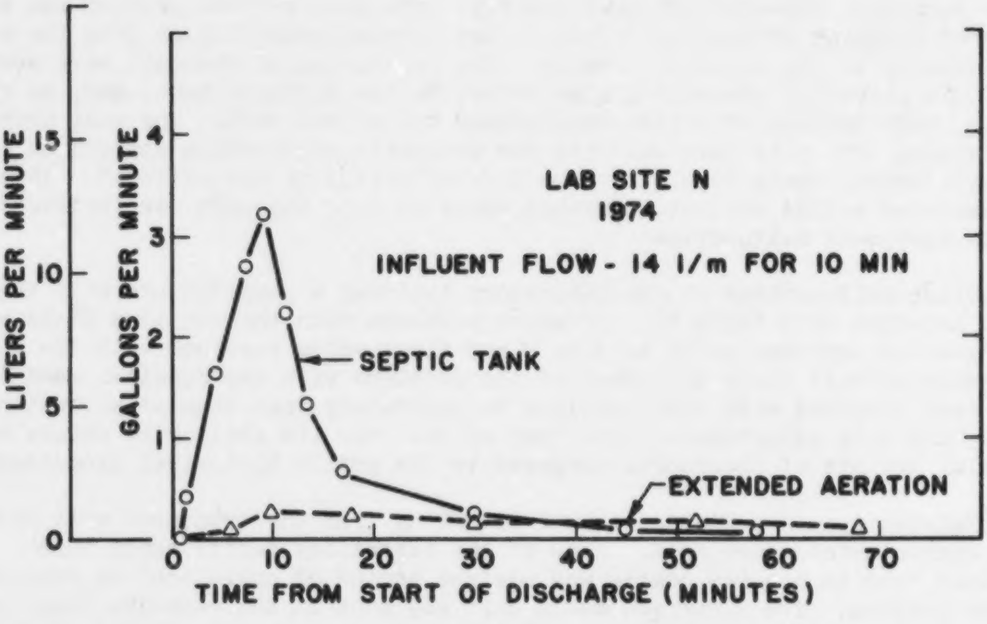


Figure A-42. Effluent flow response upon receiving a simulated clotheswasher discharge, laboratory site N, 1974.



a. Bath discharge.



b. Shower discharge.

Figure A-43. Effluent flow response upon receiving a simulated bath and shower discharge, laboratory site N, 1974.

The extended aeration unit at site N was designed especially to achieve positive solids separation and return. Not only did it dampen flow peaks to the clarifier but the clarifier was equipped with plate settlers and a positive, air-lift pumped solids return. Other configurations studied included upflow clarifiers and gravity sludge return. The contents of the rotating disks unit clarifier (site N) were automatically returned to the septic tank on a daily basis by the use of a preset timing valve.

Finally, the submerged media system employed no clarification step. Solids in the waste apparently adhered to the solid media and through circulation patterns set up in the unit by the mechanical aerator, sloughed solids moved downward into the holding tank below and adjacent to the media section.

Malfunctions--With the exceptions of the extended aeration unit at site M and the submerged media unit at site N, all the aerobic units experienced some type of malfunctioning which adversely affected effluent quality. The most serious malfunctioning occurred with the rotating disk unit (site N). Four mechanical failures (broken shaft, electrical short and pump failure twice) occurred within a one-year period. In addition, the pump clogged twice, the dump valve clogged once and the first stage of plastic disks became completely unglued. When properly operating, this unit was capable of producing a high quality effluent in terms of BOD_5 and suspended solids; however, these mechanical breakdowns caused the unit to revert to a septic tank-type of treatment.

The extended aeration unit at site N also experienced several malfunctions which seriously affected effluent quality. The most serious problem was a frequent plugging of the air lift line that pumped mixed liquor from the aeration chamber to the settling chamber. The hydraulics of the unit were such that this prevented adequate sludge return to the aeration tank, and, as a result, high amounts of solids were washed out of the unit. The only mechanical failure for this unit was with the automatic valve which controlled the air lift pumps, again resulting in a loss of solids in the effluent. Thus, the improved solids separation method designed into the unit was ineffective due to equipment malfunctions.

Other malfunctions in the laboratory included a pump failure with the batch aeration unit (site N), corrosion problems with the rotating disks and the extended aeration units at site M and float valve problems with the chemical addition unit (site M). Most of the problems with the chemical addition unit were involved with the inability to accurately feed alum at a constant rate. The only advantage of this type of unit was its ability to remove substantial amounts of phosphorus compared to the purely biological processes.

Maintenance--A periodic removal of solids from the treatment unit was the most common maintenance need. None of the laboratory septic tanks were operated long enough (19 months was maximum period of operation) to require sludge pumping. The submerged media unit (19 months), the rotating disks unit at site N (15 months), and the extended aeration unit at site M (13 months) also required no solids removal. The chemical addition unit required pumping after 9 months of operation. The average sludge production was about 0.1 kilograms per cubic meter of wastewater.

The batch aeration unit at site M demonstrated a gradual build-up of suspended solids under aeration (from about 3000 mg/L to 6000 mg/L over a 5 month period) but a premature washout of these solids prevented an accurate calculation of the sludge production. At site N, the batch unit was preceded by a septic tank so the build-up of solids was much more gradual (only 6 grams per cubic meter of wastewater treated). The mixed liquor suspended solids reached a level of 4400 mg/L after 16 months of operation. The sludge volume index (SVI) ranged from 60 near the end of testing to 360 near the beginning of testing.

The suspended solids content of the extended aeration unit at site N built up steadily (0.07 kilograms per cubic meter of wastewater) until it began malfunctioning and the solids began washing out. The solids level at this time was about 7500 mg/L. The SVI fluctuated between 60 and 120 during the period of testing. The manufacturer suggested pumping the aeration tank when the solids level reached 9000 mg/L as estimated by an Imhoff cone settling test. This level was not attained during this study.

The primary routine maintenance necessary with the chemical addition unit was the replacement of the aluminum sulfate solution (the manufacturer recommended adding 1.5 L of 50% alum solution per m^3 wastewater). This requirement necessitates either a large chemical storage tank or more frequent site visits (for example, at site M, about 30 L of alum were used per month for a flow of 0.66 m^3/day).

Other routine maintenance generally fell into two categories, parts lubrication and unit cleaning. Only the rotating disks units (both sites) and the extended aeration unit (site N) required periodic lubrication, about 1 or 2 times per year. The remaining mechanical units were equipped with factory sealed bearings. The manufacturers of the extended aeration unit (site N) and the rotating disks units were the only ones to recommend periodic cleaning as part of their maintenance plan. Other manufacturers recommended periodic inspection to ensure the satisfactory operation of the mechanical components. Unit cleaning involved the removal of any floating debris and washing of the clarifier walls to remove solids adhering to them. The recommended frequency of maintenance was every 3 months for the extended aeration unit and every 6 months for the rotating disks units.

Effluent quality and variability--The effluent quality data from the lab units are summarized in Tables A-110 and A-111. Because the wastewater was simulated, caution should be used in interpreting the actual numbers. The values for the different parameters between the various units should be used for comparison only.

The following observations can be made concerning the effluent data obtained at laboratory site M (Table A-110):

1. The aerobic units produced effluents with significantly lower levels of BOD_5, COD and suspended solids than the septic tank effluents (on the basis of 95% confidence intervals).

TABLE A-110. EFFLUENT QUALITY* - LABORATORY SITES

Site and Dates	Laboratory Site M February 1973 to August 1973					
Parameter and Statistics	Septic Tank 2.3 m^3	Septic Tank 1.1 m^3	Chemical Addition	Rotating Disks	Extended Aeration	Batch Aeration
BOD_5 (unfiltered), mg/L						
Mean (# of Samples)	125(30)	91(33)	62(32)	38(28)	53(27)	17(29)
Coeff. of Variation	0.13	0.12	0.13	0.17	0.17	0.31
95% Conf. Int.	101-162	74-112	51-77	30-49	40-69	12-24
Range	25-410	38-300	26-175	9-92	14-300	3-120
COD, mg/L						
Mean (# of Samples)	270(35)	224(36)	152(37)	51(35)	67(34)	38(35)
Coeff. of Variation	0.08	0.09	0.08	0.11	0.12	0.17
95% Conf. Int.	240-330	170-235	130-175	46-62	61-87	29-45
Range	78-750	44-500	74-350	19-110	28-308	10-240
Suspended Solids, mg/L						
Mean (# of Samples)	94(30)	77(30)	90(33)	15(32)	41(27)	18(32)
Coeff. of Variation	0.15	0.11	0.14	0.32	0.20	0.21
95% Conf. Int.	72-120	64-92	72-115	11-21	32-55	15-23
Range	21-320	36-175	34-320	0-44	6-180	4-44
Total Nitrogen, mg/L						
Mean (# of Samples)	53(23)	39(28)	39(27)	42(27)	22(25)	42(26)
Coeff. of Variation	0.09	0.18	0.18	0.09	0.11	0.11
95% Conf. Int.	44-62	30-50	30-50	36-48	19-26	36-50
Range	27-104	14-97	14-94	14-73	15-74	12-48

* All statistical computations are based on a log-normal distribution of data. Influent wastewater simulated with characteristics as outlined in Table A-106.

TABLE A-111. EFFLUENT QUALITY - LABORATORY SITES

Site and Dates / Parameter and Statistics	Laboratory Site N							
	$4\ m^3$ Septic Tank 1/30/75-3/22/76	$2\ m^3$ Septic Tank 4/28/75-3/22/76	Batch Aeration 1/30/75-2/2/76	Extended Aeration 1/30/75-3/22/76	Rotating Disks 1/30/75-11/25/75	Submerged Media 1/30/75-3/22/76	All Septic Tanks	All Aerobic Units
BOD_5 (unfiltered)								
Mean (# of Samples)	53(85)*	58(63)*	20(65)*	20(80)*	17(64)*	-	55(148)*	19(209)*
Coeff. of Variation	0.10	0.16	.25	.35	0.26		0.12	0.29
95% Conf. Int.	48-57	50-68	17-24	16-25	14-21		51-59	17-21
Range	25-180	12-310	5-150	2-430	5-80		12-310	2-430
BOD_5 (filtered)								
Mean (# of Samples)	34(85)*	26(63)*	8(65)*	5(80)*	7(63)*	-	30(148)*	6(208)*
Coeff. of Variation	0.11	0.22	0.33	0.39	0.36		0.16	0.35
95% Conf. Int.	32-38	22-31	7-9	4-6	6-8		27-33	5.5-6.5
Range	7-141	6-110	3-65	1-25	2-40		6-141	1-65
COD								
Mean (# of Samples)	130(89)	145(66)	57(70)	72(82)	52(69)	53(84)	136(155)	59(305)
Coeff. of Variation	0.06	0.10	0.14	0.18	0.12	0.15	0.08	0.15
95% Conf. Int.	125-140	128-165	49-65	61-86	46-58	47-60	134-138	58-60
Range	50-250	35-815	15-195	15-980	21-175	10-160	35-815	10-980
Total Solids								
Mean (# of Samples)	765(87)	855(67)	880(68)	810(80)	810(68)	755(85)	804(154)	824(301)
Coeff. of Variation	0.15	0.17	0.22	0.20	0.14	0.15	0.16	0.18
95% Conf. Int.	740-790	820-890	835-925	825-905	780-835	730-780	783-825	807-840
Range	480-1230	540-1420	450-1200	300-1500	500-1050	450-1090	470-1420	300-1500
Volatile Solids								
Mean (# of Samples)	200(82)	255(63)	290(67)	265(76)	240(67)	185(78)	234(145)	243(288)
Coeff. of Variation	0.28	0.44	0.35	0.42	0.24	0.33	0.37	0.35
95% Conf. Int.	190-215	230-285	265-315	240-290	225-250	170-200	220-248	233-253
Range	50-390	110-790	60-500	11-800	100-405	60-360	50-800	11-800

(continued)

TABLE A-111 (continued)

Units	Laboratory Site N							
Parameter and Statistics	4 m^3 Septic Tank	2 m^3 Septic Tank	Batch Aeration	Extended Aeration	Rotating Disks	Submerged Media	All Septic Tanks	All Aerobic Units
Suspended Solids, mg/L								
Mean (# of Samples)	27(86)*	50(65)*	26(69)*	46(81)*	16(67)*	15(85)*	35(151)*	23(302)*
Coeff. of Variation	0.16	0.20	0.20	0.24	0.20	0.34	0.18	0.25
95% Conf. Int.	24-30	41-60	22-30	38-57	14-18	12-18	32-39	21-25
Range	7-175	8-250	7-120	11-1230	3-80	<1-64	7-175	0-1230
Volatile Suspended, mg/L								
Mean (# of Samples)	18(85)*	38(64)*	17(69)*	33(81)*	11(67)*	9(85)*	25(149)*	15(302)*
Coeff. of Variation	0.16	0.24	0.25	0.27	0.28	0.48	0.20	0.32
95% Conf. Int.	17-20	31-47	15-21	27-41	9-12	7-12	23-28	14-17
Range	5-48	5-250	4-100	6-810	2-50	<1-40	5-250	0-810
Total Nitrogen, mg-N/L								
Mean (# of Samples)	51(57)	56(35)	43(52)	40(59)	40(57)	28(55)	53(92)	37(227)
Coeff. of Variation	0.28	0.33	0.49	0.40	0.45	0.58	0.30	0.48
95% Conf. Int.	47-55	49-62	37-49	36-44	35-45	23-32	50-56	35-39
Range	11-87	31-112	2-84	4-80	2-117	6-90	11-112	2-117
Ammonia Nitrogen, mg-N/L								
Mean (# of Samples)	39(57)	31(35)	2(52)	(59)	7(57)	1(55)	36(92)	2.5(227)
Coeff. of Variation	0.48	0.38	2.9	-	1.75	3.78	0.45	2.60
95% Conf. Int.	34-44	27-35	0.4-3.8	.01-0.1	4-10	<0.1-2.3	33-39	1.6-3.3
Range	<0.1-83	10-55	<0.1-42	<0.1-1.8	<0.1-51	<0.1-24	<0.1-83	<0.1-51
Nitrate Nitrogen, mg-N/L								
Mean (# of Samples)	3(57)	12(35)	39(52)	36(59)	31(57)	19(55)	6.4(92)	31(227)
Coeff. of Variation	4.2	1.0	0.58	0.45	0.51	0.59	1.92	0.55
95% Conf. Int.	<0.1-5	8-16	32-45	32-40	27-36	16-22	3.8-8.9	29-33
Range	<0.1-60	<0.1-54	1-84	2-80	<0.1-71	<0.1-52	<0.1-60	<0.1-80

(continued)

TABLE A-111 (continued)

Units	Laboratory Site N							
Parameter and Statistics	4 m^3 Septic Tank	2 m^3 Septic Tank	Batch Aeration	Extended Aeration	Rotating Disks	Submerged Media	All Septic Tanks	All Aerobic Units
Total Phosphorus, mg-P/L								
Mean (# of Samples)	34(57)	38(35)	32(52)	32(59)	32(58)	36(59)	36(92)	33(228)
Coeff. of Variation	0.28	0.25	0.36	0.34	0.23	0.31	0.26	0.31
95% Conf. Int.	32-37	35-42	28-35	29-35	30-33	33-38	34-38	32-34
Range	13-55	21-55	10-58	13-72	14-53	14-66	13-55	10-72
Orthophosphorus, mg-P/L								
Mean (# of Samples)	26(57)	28(35)	25(52)	25(59)	26(58)	26(59)	27(92)	26(228)
Coeff. of Variation	0.26	0.24	0.32	0.13	0.22	0.29	0.25	0.24
95% Conf. Int.	24-28	25-30	23-28	23-27	25-28	24-28	26-28	25-27
Range	5-42	18-47	8-48	5-45	5-40	12-46	5-47	5-48

* Statistical computations based on log-normal distribution of the data.
Influent wastewater simulated with characteristics as outlined in Table A-106.

2. The variability of data as indicated by the coefficient of variation is greater for the aerobic units than for the septic tanks. The range of data was large for all units.

3. The extended aeration unit, which had cycled aeration, removed a substantial portion of the total nitrogen (over 50%).

4. The batch aeration unit produced the highest quality effluent in terms of BOD_5, COD, and suspended solids.

5. Although the chemical addition unit was able to remove about 30% of the BOD_5 and COD from the 1.1 m^3 septic tank, the level of suspended solids increased by 30%.

At laboratory site N (Table A-111) the following observations concerning the effluent quality data are noted:

1. The aerobic units as a group produced effluents with significantly lower levels of BOD_5, COD, suspended solids, and total nitrogen as compared to the septic tanks as a group.

2. There was no significant difference in total solids and total phosphorus levels between the aerobic units and the septic tanks.

3. The BOD_5 values for the submerged media unit are not presented because they were collected at a point following ultraviolet irradiation. Subsequent tests showed that this had a significant effect of reducing BOD_5 levels. The COD's were not affected.

4. Except for the parameters of total and volatile solids, the variability of effluent quality data (coefficient of variation) was greater for the aerobic units than for the septic tanks.

5. Most of the nitrogen (≃ 67%) from the septic tanks was in the form of ammonia-nitrogen.

6. Most of the nitrogen (≃ 83%) from the aerobic units was in the form of nitrate-nitrite-nitrogen.

7. The submerged media unit's total nitrogen level was about 47% lower than the septic tank levels. All the aerobic units had lower total nitrogen levels than the septic tanks.

8. Most of the phosphorus (≃ 77%) from both the septic tanks and the aerobic units was in the form of orthophosphate.

9. Most of the BOD_5 (≃ 68%) from the aerobic units was in a filterable form. Much of the BOD_5 (≃ 55%) from the septic tanks is in a soluble form.

10. The rotating disks and the submerged media units had the lowest levels of suspended solids (≃ 16 mg/L) in their effluents.

Field Installations--

Operation--Of the field units, the septic tanks are the simplest to operate and the easiest to install. Since their normal operation has been previously discussed, it will not be covered here. Tables A-104 and A-107 campare tank sizes and periods of operation.

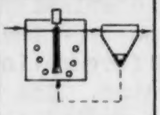
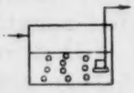
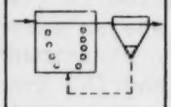
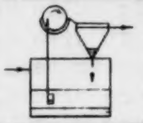
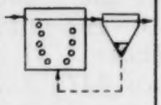
The aerobic units are compared as to their normal operation and maintenance along with noted malfunctions in Table A-112. As with the laboratory units, the aerobic units varied in terms of their complexity and their components. The unit at site C was an extended aeration type that had the following features: (1) septic tank pretreatment, (2) on-off operation of aerator, (3) air lift sludge return, and (4) draft-tube aerator. The unit was operated for a total of over 2-1/2 years. During this time period, several operational problems were noted. Probably the most important was a continual build-up of solids on the clarifier surface. These solids would occasionally wash over the effluent weir, resulting in poor effluent quality. Another problem was sludge loss, which appeared to be related to the use of toilet bowl deodorizers. After several months the mixed liquor finally acclimatized and returned to normal levels. Sludge bulking occurred one winter, with this condition lasting for over 4 months. Toward the end of testing, the wiring to the aerator had to be replaced.

The batch aeration unit at site G was similar to the one installed in the laboratory. The mechanical components were obtained from the manufacturer and installed in a 4.5 cubic meter (1200 gal) septic tank. Improper wiring during installation led to almost immediate failure of the electronics of the system. The repaired unit was then operated and monitored for over 3 years. During this period the following problems were noted: (1) the pump clogged with toilet paper twice; (2) the pump failed once; (3) the unit experienced excessive foaming during start-up; and (4) a faulty time clock was repaired. Although it did not involve the batch aeration process, it is important to note that the sump pump following this unit failed twice and the electrical wiring in the sump had to be completely replaced due to corrosion problems after four years.

Of all the aerobic units installed, the one at site H was operated the longest (over 4-1/2 years). After 28 months of operation, septic tank pretreatment was included as part of the treatment system. Other features of this extended aeration unit were an air lift sludge return and a surface skimmer and scum return on the final clarifier. The mechanical problems which occurred were air line failure (3 times) and skimmer motor failure (4 times). Prior to the use of septic tank pretreatment, the skimmer continually clogged due to large accumulations of floating debris (grease and garbage) on the surface of the clarifier. Once the septic tank was introduced, this ceased to be a problem. Sludge bulking was a serious problem with this unit. When the temperature of the mixed liquor dropped below 15°C, filamentous bulking would occur (this happened every winter during the period of operation). Only when the temperature rose back to about 20°C did bulking subside.

The rotating disks unit installed at site I was similar to the unit installed at laboratory site N. This unit was continually plagued with mechanical problems; therefore, the effluent quality data were not reported. It is

TABLE A-112. PROCESS COMPARISONS - OPERATION AND MAINTENANCE
Aerobic Field Units

TREATMENT PROCESS	EXTENDED AERATION	BATCH AERATION	EXTENDED AERATION	ROTATING DISKS	EXTENDED AERATION
Field Site	C	G	H	I	J
Sketch of Process					
Primary Biological State	Aerobic, Suspended Growth	Aerobic, Suspended Growth	Aerobic Suspended Growth	Aerobic, Attached Growth	Aerobic Suspended Growth
Mechanical Components	Aerator Motor, Timer Controlled	Air Compressor, Submersible Pump, Timer Controlled, Alarm	Blower and Motor, Skimmer Motor	Gear Motor, Pump, Dump Valve, Timer Controlled, Alarm	Blower and Motor, Alarm
Volume	2.8 m^3	about 2.0 m^3	3.0 m^3	Disk Module - 0.24 Septic Tank - 2.0 to 4.0 m^3	2.0 m^3
Flow Character	Continuous Flow, Preceded by Septic Tank	Batch Discharge	Continuous Flow, Preceded by Septic Tank After Nov. 1974	Equalized, Constant Flow Thru Disks Module	Continuous Flow, Preceded by Septic Tank
Solids Separation	Gravity Settling, Upflow Clarifier,	Aeration and Settling Occur in Same Chamber	Gravity Settling, Air Lift Sludge Return, Scum Return	Upflow Clarifier, Contents Returned to Septic Tank Daily	Gravity Settling, Upflow Clarifier
Time Cycle	Aerator ON 15 min, OFF 15 min	Aerator ON 20 hr, OFF 4 hr, Pump-Out After 3 1/2 hr of Settling	None before Sept. 1974, Aerator ON 1 hr, OFF 1 hr, Sept. 1974 to Oct. 1975	Dump Valve Opens Once per Day	None
Period of Operation	December 1972-April 1975	August 1972-November 1975	July 1972-December 1976	August 1974-November 1975	February 1975-December 1976
Measured Power Consumption	Not Measured	5.8 kwh/d	Not Measured	3.8 kwh/d	Not Measured
Power Input/Vol.	—	0.14 kw/m^3	—	0.66 kw/m^3	—
Required Routine Maintenance	Solids Removal, Aerator Lubrication	Solids Removal	Solids Removal, Service Blower, Clean Unit	Solids Removal Parts Lubrication, Clean Unit	Solids Removal
Malfunctions	Faulty Wiring at Installation Sludge Bulking-1 Winter Poor Settling Sludge	Pump Failed, Pump Clogged, Improper Wiring at Installation, Time Clock Replaced	Frequent Skimmer Clogging, Air Line Failed 3 Times, Skimmer Motor Failed 4 Times Sludge Bulking-3 Winters	Two Shafts Broke Pump Failed, Motion Sensor Failed, Drain Lines Clogged	Air Line Failed Sludge Return Clogged by Sediment

included in the discussion, however, because it gives an example of the problems which can occur. Although the unit was only operated for 15 months, the following malfunctions occurred: (1) the shaft on one of the disk modules broke twice; (2) the pump failed once; (3) the drain lines and the pump occasionally clogged; (4) the motion sensor corroded and failed, (5) the electronics were damaged when the unit was apparently struck by lightning; and (6) the bondings between the individual disks were so poor that they all became separated.

The unit at site J was also an extended aeration type, and it was operated for a 2-year period. The only problems noted were: (1) the vibration of the unit wore a hole in the air line and (2) grit in the bottom of the unit prevented adequate sludge return. This unit performed the best of all the field units; however, it should be noted that it was preceded by septic tank treatment and was only loaded at a rate of 0.25 m^3/d (65 gpd) or about 1/6 of the manufacturers design flow.

Maintenance--Routine maintenance of the field units fell into two categories, solids removal and equipment servicing. Although some manufacturers suggest that excess solids wasting is not necessary, the unit should be pumped every 8 to 12 months. Typically, mixed liquor suspended solids (MLSS) concentrations built up over a period of time until they reached 8000 to 10000 mg/L. When concentrations reached this point, however, solids were lost in the effluent until the MLSS returned to 3000 to 6000 mg/L. System C was pumped twice at intervals of 11 and 15 months, system G at intervals of 15 and 11 months and system H at intervals of 24 and 10 months. Severe sludge bulking with unit H had caused many of the solids to be lost in the effluent, thus increasing the first pumping interval. System J did not require pumping during the 24 month operating period because it had received such a low organic loading.

During the period of monitoring, none of the septic tanks required pumping. Site E was monitored the longest (over 4 years), and site F the shortest (17 months).

Equipment servicing included greasing the aerator bearings on unit C, and cleaning the air inlet to the blower on the aerobic unit at site J. Other routine inspection tasks included such things as checking for V-belt wear at sites H and J, checking alarms (sites G and J), determining the settling properties of the sludge and checking the integrity of the components. The maintenance schedules for mechanical components suggested by the manufacturers appeared to be adequate.

Only the manufacturers of the extended aeration unit at site H and the rotating disks unit recommended periodic unit cleaning as part of the routine maintenance. Suggested cleaning of the extended aeration unit included removal of scum and floating debris from the clarifier, cleaning of the scum return line, cleaning the weir area, and general clean up around the exterior of the unit. The rotating disk unit manufacturer suggested washing down the walls of the clarifier, flushing all drain lines, and cleaning the clarifier weir area on a semi-annual basis.

Effluent quality and variability--The results from monitoring the field septic tank effluents appear in Table A-113. As would be expected for this type of treatment process, the effluent quality was relatively poor with respect to organic matter, indicator organisms and nutrients. The suspended solids values were indicative of adequate solids separation which again would be expected since septic tanks are designed for solids removal. The following observations are also noted:

1. There were significant differences on the basis of 95% confidence intervals in the average BOD_5 and COD levels between units ranging from 57 mg/L (site J) to 272 mg/L (site C) for BOD_5 and 208 mg/L (site J) to 542 mg/L (site C).

2. For most sites, there was no significant difference in the total suspended solids (TSS) levels between units (site J had significantly less TSS than sites A and C). The average level was 49 mg/L for all units as a group with range of averages from 34 mg/L (site J) to 69 mg/L (site A).

3. The level of total nitrogen varied considerably between units (from 26 mg/L at site E to 76 mg/L at site C). When considering all the septic tanks as a group, 69% of the nitrogen was in the form of ammonia (ranged from 60% at site C to 82% at site D).

4. There was essentially no difference in the total phosphorus levels between the various septic tanks. Approximately 85% of the phosphorus was in the form of orthophosphate.

5. There was no difference in the levels of indicator bacteria from the various units. The average fecal coliform level was 5×10^6 organisms per liter ($10^{6.7}$ organisms per liter) and the level of fecal streptococci was 4×10^4 ($10^{4.6}$) organisms per liter.

Site E (two tanks in series) was monitored in three phases. It is interesting to note that although the average flow to the tanks was almost three times less than normal during the first phase, there was essentially no difference in effluent qualities. Two-tanks-in-series (sites E and J) appear to produce slightly higher effluent quality in terms of organic matter and suspended solids than do the single chambered tanks.

The performance in terms of effluent quality of the five aerobic units is presented in Table A-114. Also included in this table is a summary of the grouped septic tank data for comparisons. It is immediately apparent that the aerobic units produced a higher degree of treatment than septic tanks but that the periodic malfunctions and upsets mentioned earlier resulted in substantial variability in effluent quality. It is important to note that the effluent suspended solids level of the aerobic units although lower than that of septic tanks, was not significantly different on the basis of the 95% confidence intervals. The following observations concerning the aerobic units are also noted:

1. The extended aeration unit at site J produced the best effluent in terms of organic matter and suspended solids, but the hydraulic and

TABLE A-113. SEPTIC TANK EFFLUENT QUALITY - FIELD SITES

Site and Dates	A	B	C	D		E		F	J	All Sites
	May 1972-May 1974	May 1972-May 1974	Jul 1972-Jun 1974	Aug 1972-Jun 1974	Oct 1972-Jun 1974	Aug 1974-Mar 1975	Mar 1975-Dec 1976	Nov 1972-Jun 1974	Jun 1976-Dec 1976	May 1972-Dec 1976
Parameter and Statistics	0.28 m^3/d (75 gpd)	0.47 m^3/d (125 gpd)	0.93 m^3/d (245 gpd)	1.2 m^3/d (315 gpd)	3.3 m^3/d (860 gpd)	1.1 m^3/d (290 gpd)	1.2 m^3/d (330 gpd)	0.57 m^3/d (150 gpd)	0.64 m^3/d (170 gpd)	0.28-3.3 m^3/d
BOD_5 (unfiltered), mg/L										
Mean (# of Samples)	131(19)	176(19)	272(14)	127(19)	120(18)	123(13)	109(13)	122(23)	57(12)	138(150)
Coeff. of Variation	0.52	0.34	0.34	0.37	0.43	0.56	0.54	0.30	0.27	0.42
95% Conf. Int.	98-165	147-206	216-327	104-149	94-145	81-165	73-145	106-138	47-66	129-147
Range	20-260	87-350	131-480	54-210	23-223	22-235	7-215	33-175	40-85	7-480
BOD_5 (filtered), mg/L										
Mean (# of Samples)	107(18)	141(17)	196(12)	97(19)	98(19)	87(13)	69(7)	95(22)	42(3)	109(130)
Coeff. of Variation	0.54	0.45	0.40	0.45	0.46	0.64	0.64	0.32	0.18	0.47
95% Conf. Int.	79-136	108-173	146-247	76-118	76-120	53-121	16-122	81-108	23-61	100-118
Range	7-220	46-300	75-330	22-175	13-210	10-175	2-170	38-150	35-50	7-330
COD, mg/L										
Mean (# of Samples)	325(20)	361(20)	542(14)	291(19)	294(20)	280(10)	271(13)	337(24)	208(12)	327(152)
Coeff. of Variation	0.40	0.26	0.22	0.40	0.40	0.54	0.48	0.17	0.28	0.33
95% Conf. Int.	265-386	317-406	473-612	235-347	239-349	173-387	192-351	313-316	172-244	310-344
Range	114-650	66-540	342-780	157-686	96-545	65-520	25-460	215-440	157-350	25-780
Total Solids, mg/L										
Mean (# of Samples)							1060(13)		1340(12)	
Coeff. of Variation							0.36		0.37	
95% Conf. Int.							830-1290		1030-1650	
Range							410-1800		920-2310	
Volatile Solids, mg/L										
Mean (# of Samples)							250(13)		260(12)	
Coeff. of Variation							0.41		0.34	
95% Conf. Int.							190-310		200-310	
Range							70-420		120-410	

(continued)

TABLE A-113 (continued)

Parameter and Statistics / Site and Dates	A	B	C	D		E		F	J	All Sites
	May 1972-May 1974	May 1972-May 1974	Jul 1972-Jun 1974	Aug 1972-Jun 1974	Oct 1972-Jun 1974	Aug 1974-Mar 1975	Mar 1975-Dec 1976	Nov 1972-Jun 1974	Jun 1976-Dec 1976	
Suspended Solids, mg/L										
Mean (# of Samples)	69(18)*	44(18)*	68(15)*	52(18)*	51(19)*	48(11)*	39(13)*	48(24)*	34(12)*	49(148)*
Coeff. of Variation	0.20	0.14	0.13	0.18	0.15	0.24	0.18	0.16	0.07	0.16
95% Conf. Int.	46-104	34-58	50-92	36-72	39-68	26-89	26-57	37-61	29-40	44-54
Range	20-695	18-124	34-380	12-291	18-140	10-256	14-135	11-154	23-50	10-695
Volatile Suspended, mg/L										
Mean (# of Samples)	43(18)*	32(18)*	56(15)*	32(18)*	40(19)*	37(11)*	28(13)*	33(24)*	28(12)*	35(148)*
Coeff. of Variation	0.17	0.15	0.19	0.21	0.18	0.22	0.24	0.16	0.08	0.18
95% Conf. Int.	31-59	25-41	36-86	22-45	29-55	21-63	17-45	26-43	23-33	32-39
Range	16-112	12-68	16-320	9-154	10-132	7-124	5-105	10-76	17-48	5-320
Total Nitrogen, mg-N/L										
Mean (# of Samples)	50(13)	58(11)	76(8)	40(10)	32(11)	26(9)	27(9)	57(11)	36(12)	45(99)
Coeff. of Variation	0.58	0.33	0.26	0.18	0.35	0.42	0.84	0.37	0.07	0.40
95% Conf. Int.	33-68	45-70	60-93	35-45	24-39	17-34	10-45	43-71	34-37	41-49
Range	10-125	26-103	62-111	28-50	16-50	16-51	9-86	34-98	31-40	9-125
Ammonia Nitrogen, mg-N/L										
Mean (# of Samples)	34(15)	42(13)	46(11)	33(11)	20(13)	21(11)	29(9)	38(13)	26(12)	31(108)
Coeff. of Variation	0.54	0.38	0.52	0.25	0.48	0.38	0.79	0.41	0.22	0.46
95% Conf. Int.	23-45	33-52	30-61	28-39	14-26	15-26	7-30	29-48	22-29	28-34
Range	0.1-68	4-63	8-91	16-46	1-32	11-35	7-56	0.9-62	10-31	0.1-91
Nitrate Nitrogen, mg-N/L										
Mean (# of Samples)	0.7(16)	0.5(15)	0.6(10)	0.4(13)	0.2(15)	0.3(11)	0.1(9)	0.8(13)	0.2(12)	0.4(114)
Coeff. of Variation	4.0	2.7	2.0	0.77	0.9	-	0.7	9.8	1.21	6.7
95% Conf. Int.	0.4-1.4	0.2-1.5	0.3-1.3	0.2-0.6	0.1-0.4	-	0-0.2	0.9-2.4	0-0.3	0-0.9
Range	0.3-74	0-30	0.2-6.6	0-1.1	0-0.6	-	0-0.4	0.1-30	0.1-0.8	0-74

(continued)

TABLE A-113 (continued)

Parameter and Statistics / Site and Dates	A	B	C	D	E			F	J	All Sites
	May 1972-May 1974	May 1972-May 1974	Jul 1972-Jun 1974	Aug 1972-Jun 1974	Oct 1972-Jun 1974	Aug 1974-Mar 1975	Mar 1975-Dec 1976	Nov 1972-Jun 1974	Jun 1976-Dec 1976	
Total Phosphorus, mg-P/L										
Mean (# of Samples)	12(15)	14(13)	31(6)	11(14)	11(14)	11(11)	12(10)	12(6)	16(10)	13(99)
Coeff. of Variation	0.47	0.30	0.94	0.37	0.45	0.43	0.48	0.14	0.08	0.34
95% Conf. Int.	9-16	12-17	0.4-62	9-13	8-14	8-14	8-16	10-13	15-17	12-14
Range	0.7-20	6-20	16-90	4-18	3-20	6-22	4-24	9-14	15-18	0.7-90
Orthophosphorus, mg-P/L										
Mean (# of Samples)	11(12)	14(11)	14(3)	10(11)	10(13)	10(11)	8(10)	10(6)	15(12)	11(89)
Coeff. of Variation	0.44	0.32	0.2	0.45	0.50	0.46	0.61	0.26	0.13	0.36
95% Conf. Int.	8-14	11-17	7-21	7-13	7-14	7-13	5-12	8-13	13-16	10-12
Range	4-18	6-20	11-17	3-15	2-19	4-19	3-20	6-14	10-17	3-20
Total Coliforms, log$_{10}$#/L										
Mean (# of Samples)						7.0(12)	6.4(10)		7.0(14)	
Coeff. of Variation						0.08	0.19		0.08	
95% Conf. Int.						6.7-7.2	5.7-7.2		6.7-7.3	
Range						6.3-7.4	4.8-7.5		6.3-7.9	
Fecal Coliforms, log$_{10}$#/L										
(Mean (# of Samples)	6.5(23)	6.6(21)	7.4(9)	7.0(19)	6.4(19)	6.8(14)	6.3(10)	6.5(22)	6.8(14)	6.7(151)
Coeff. of Variation	0.24	0.23	0.36	0.19	0.45	0.09	0.24	0.14	0.07	0.24
95% Conf. Int.	6.1-6.8	6.2-7.0	5.5-7.4	6.7-7.4	5.6-7.1	6.6-7.0	5.4-7.2	6.3-6.7	6.6-7.0	6.4-7.0
Range	3.7-7.2	4.8-8.2	3.9-7.4	5.2-7.8	2.0-7.6	6.1-7.3	4.0-7.6	5.9-7.9	6.2-7.4	2.0-8.2
Fecal Strep, log$_{10}$#/L										
Mean (# of Samples)	3.4(23)	4.6(21)	6.1(11)	5.3(19)	4.3(20)	4.5(14)	5.0(10)	4.4(23)	5.2(14)	4.6(155)
Coeff. of Variation	3.0	0.65	0.21	0.40	0.81	0.40	0.36	0.90	0.20	1.0
95% Conf. Int.	2.8-4.0	4.1-5.1	5.7-6.6	4.8-5.7	3.8-4.8	4.1-4.8	4.0-6.0	3.9-4.9	4.8-5.7	3.9-5.3
Range	1.3-6.7	2.3-6.7	5.2-7.0	3.0-6.8	2.0-7.2	3.0-5.3	3.9-7.6	2.0-6.4	4.0-6.4	2.0-7.6
Ps. aeruginosa, log$_{10}$#/L										
Mean (# of Samples)							5.1(8)		3.9(8)	
Coeff. of Variation							0.35		0.14	
95% Conf. Int.							3.9-6.3		3.5-4.2	
Range							3.3-7.4		3.3-4.4	
*										

*Log normal distribution.

TABLE A-114. AEROBIC UNIT EFFLUENT QUALITY - FIELD SITES

Site and Dates	C	G	H		J	All Sites	Septic Tanks - All Sites
	Jul 1972- Jun 1974	Nov 1972- Jun 1974	Aug 1973- Feb 1975	Mar 1975- May 1976	Jun 1976- Dec 1976	Jul 1972- Dec 1976	May 1972- Dec 1976
Parameter and Statistics	0.93 m^3/d (245 gpd)	0.64 m^3/d (170 gpd)	0.39 m^3/d (102 gpd)	0.53 m^3/d (140 gpd)	0.25 m^3/d (65 gpd)	0.25 - 0.93 m^3/d	0.28 - 3.3 m^3/d
BOD_5 (unfiltered), mg/L							
Mean (# of Samples)	55(19)	55(19)	26(48)	48(15)	6(11)	37(112)	138(150)
Coeff. of Variation	0.80	0.84	0.20	0.67	0.89	0.78	0.42
95% Conf. Int.	34-76	33-78	21-31	30-65	2-10	32-42	129-147
Range	4-160	5-208	6-86	10-145	2-18	2-208	7-480
BOD_5 (filtered), mg/L							
Mean (# of Samples)	20(18)	20(17)	9(45)	22(14)		15(94)	109(130)
Coeff. of Variation	0.65	0.70	0.41	1.35		0.94	0.47
95% Conf. Int.	14-26	13-27	7-12	5-39		12-18	100-118
Range	2-50	1-45	1-60	3-120		1-120	7-330
COD, mg/L							
Mean (# of Samples)	158(20)	159(20)	82(50)	89(15)	71(11)	108(116)	327(152)
Coeff. of Variation	0.45	0.37	0.12	0.51	0.51	0.40	0.33
95% Conf. Int.	125-191	132-186	20-95	66-120	47-95	100-116	310-344
Range	49-349	48-293	20-210	28-220	30-160	20-349	25-780
Total Solids, mg/L							
Mean (# of Samples)				720(15)	1210(11)		
Coeff. of Variation				0.08	0.28		
95% Conf. Int.				690-750	990-1440		
Range				600-800	920-1920		
Volatile Solids, mg/L							
Mean (# of Samples)				230(75)	210(11)		
Coeff. of Variation				0.29	0.41		
95% Conf. Int.				190-270	150-270		
Range				130-320	80-300		

(continued)

TABLE A-114 (continued)

Parameter and Statistics / Site and Dates	C	G	H		J	All Sites	Septic Tanks All Sites
	Jul 1972- Jun 1974	Nov 1972- Jun 1974	Aug 1973- Feb 1975	Mar 1975- May 1976	Jun 1976- Dec 1976		
Suspended Solids, mg/L							
Mean (# of Samples)	38(20)	65(20)	48(51)*	23(15)*	12(11)	39(117)*	49(148)*
Coeff. of Variation	0.26	0.17	0.18	0.20	0.68	(.23)	0.16
95% Conf. Int.	24-60	47-90	39-58	16-32	6-17	33-46	44-54
Range	4-156	21-252	5-164	8-45	3-31	3-252	10-695
Volatile Suspended, mg/L							
Mean (# of Samples)	31(21)*	44(20)	33(51)*	15(15)*	5(11)	27(118)*	35(148)*
Coeff. of Variation	0.28	0.23	0.22	0.33	0.61	.26	0.18
95% Conf. Int.	20-47	29-66	27-41	9-24	3-7	23-32	32-39
Range	4-16	3-144	4-136	2-40	1-10	1-144	5-320
Total Nitrogen, mg-N/L							
Mean (# of Samples)	32(11)	39(10)	39(46)	31(8)	31(12)	36(87)	45(99)
Coeff. of Variation	0.41	0.36	0.33	0.29	0.08	0.32	0.40
95% Conf. Int.	23-41	29-49	35-43	24-39	30-32	34-38	41-49
Range	16-62	17-64	20-78	15-44	26-34	15-78	9-125
Ammonia Nitrogen, mg-N/L							
Mean (# of Samples)	0.7(12)	<0.1(13)	0.4(47)	7(8)	<0.1(12)	0.9(92)	31(108)
Coeff. of Variation	11.6	0.95	3.65	1.16	-	4.2	0.46
95% Conf. Int.	<0.1-6.6	<0.1-0.2	0-1.6	0.2-14	-	0.1-1.7	28-34
Range	<0.1-60	<0.1-37	<0.1-19	<0.1-23	-	<0.1-60	0.1-91
Nitrate Nitrogen, mg-N/L							
Mean (# of Samples)	19(13)	30(12)	34(50)	22(8)	30(12)	30(95)	0.4(114)
Coeff. of Variation	0.77	0.45	0.41	0.71	0.07	0.44	6.7
95% Conf. Int.	10-28	21-38	30-38	9-34	30-31	27-33	<0.1-0.9
Range	0.3-41	3-54	0.3-72	1-39	26-33	0.3-72	<0.1-74

(continued)

TABLE A-114 (continued)

Site and Dates Parameter and Statistics	C	G	H		J	All Sites	Septic Tanks All Sites
	Jul 1972-Jun 1974	Nov 1972-Jun 1974	Aug 1973-Feb 1975	Mar 1975-May 1976	Jun 1976-Dec 1976		
Total Phosphorus, mg-P/L							
Mean (# of Samples)	25(11)	26(10)	32(41)	17(8)	9(10)	26(80)	13(99)
Coeff. of Variation	0.91	0.36	0.73	0.22	0.19	0.74	0.34
95% Conf. Int.	10-41	20-33	24-39	14-20	8-10	22-30	12-14
Range	7-90	14-43	9-140	13-24	6-12	6-140	0.7-90
Orthophosphorus, mg-P/L							
Mean (# of Samples)	17(8)	23(9)	28(41)	10(8)	8(12)	21(78)	11(89)
Coeff. of Variation	0.39	0.30	0.54	0.31	0.16	0.54	0.36
95% Conf. Int.	11-22	18-28	23-33	8-13	7-9	18-24	10-12
Range	7-24	12-35	9-51	7-16	6-10	6-51	3-20
Total Coliforms, $\log_{10}$#/L							
Mean (# of Samples)				6.2(9)	4.4(14)		
Coeff. of Variation				0.22	0.12		
95% Conf. Int.				5.3-7.1	4.2-4.6		
Range				4.0-7.7	3.8-5.3		
Fecal Coliforms, $\log_{10}$#/L							
Mean (# of Samples)	5.0(20)	4.8(21)	5.3(51)	5.2(9)	4.1(14)	5.0(115)	6.7(151)
Coeff. of Variation	0.34	0.45	0.24	0.36	0.16	0.30	0.24
95% Conf. Int.	4.7-5.3	4.4-5.2	5.1-5.4	4.0-6.4	3.8-4.4	4.7-5.3	6.4-7.0
Range	3.6-6.3	3.0-6.2	4.2-6.4	2.8-7.3	3.3-5.2	2.8-7.3	2.0-8.2
Fecal Strep, $\log_{10}$#/L							
Mean (# of samples)	4.6(19)	4.6(19)	4.4(52)	4.4(9)	3.4(14)	4.3(113)	4.6(155)
Coeff. of Variation	0.46	0.43	0.53	0.58	0.34	0.49	1.0
95% Conf. Int.	4.2-4.9	4.3-4.9	4.2-4.6	2.9-5.9	2.9-3.8	3.9-4.7	3.9-5.3
Range	3.6-6.3	3.3-5.5	2.3-6.1	2.0-6.3	2.1-4.4	2.0-6.3	2.0-7.6

* Log normal distribution.

organic loadings to it were substantially lower than normal (it received about 40% of the septic tank effluent at site J).

2. Because of the settling problems noted there is considerable difference in the average TSS levels between the various units, ranging from 12 mg/L to 65 mg/L.

3. The total nitrogen values did not show the variation between sites that the septic tanks did. When using the grouped data, 83% of the total nitrogen is in the form of nitrate.

4. Unit C which received septic tank effluent (see Table A-113) and had cycled aeration, removed over 1/2 of the total nitrogen. Unit J also received septic tank effluent but only removed about 15% of the nitrogen. Its aerator was operated continuously.

5. The total phosphorus levels varied considerably from site to site since phosphorus loading is generally a function of household laundry use which varies from home to home (Witt, 1974a). About 81% of the phosphorus is in the form of orthophosphate.

6. Except for site J, there was essentially no difference in the bacterial counts between aerobic units. There was also no difference between the fecal streptococci levels from the aerobic units versus the septic tank. The fecal coliform levels were almost 2 orders of magnitude lower in the aerobic units than in the septic tanks (10^5 versus $10^{6.7}$ per liter).

7. As indicated by the coefficient of variation, the aerobic units produced a more variable effluent quality in terms of organic matter and suspended solids.

Discussion of Results

In evaluating the laboratory and field effectiveness of selected treatment processed for onsite application, four criteria were used: (1) operation and maintenance requirements, (2) the average effluent quality produced, (3) the variability in effluent quality, and (4) total annual cost.

Operation--

The continuous operation of a treatment unit is critical to its acceptance as an alternative for onsite use. It must be simple, yet rugged and reliable, producing an effluent of predictable quality. A listing of the operational characteristics of the laboratory and field treatment units appear in Tables A-108, A-109 and A-112. Some of the important operational features of these systems are discussed below.

Installation--Proper installation is important for the successful operation of any treatment unit, and as installation becomes more complex, greater care must be taken. For aerobic units and sump pumps, the services of a qualified electrician are necessary. Two of the field units (sites C and G) suffered from improper installation. At site G, where the unit was incorrectly wired, failure was almost immediate. Only after 2 years of operation did the installation problem (faulty wiring) at site C become apparent and then only after noting a substantial increase in the homeowner's electric bill. Prior to correcting this problem, the residents received electrical shocks from their plumbing system.

Voell and Vance (1974) alluded to installation problems such as finding aerobic units completely buried and timer controls placed in inaccessible places. Glasser (1974) recommended a site visit within 10 days of installation to ensure proper operation of the mechanical components. (Several methods by which regulatory agencies can ensure proper installation are discussed in Appendix D.)

Design features--A discussion of the important design features of waste treatment systems based on the experience of investigators and plant operators over the years has been presented earlier. A discussion of some of these design features in light of experiences with units installed during this project is presented below.

Multiple tanks in series--Previous researchers have demonstrated that multi-compartment septic tanks provide better treatment than single chambered tanks of the same capacity (Ludwig, 1950; Weibel, 1955). Two field sites (E and J) incorporated tanks-in-series and did demonstrate slightly higher effluent quality (Table A-113) but the total capacities were greater (about 50% at site E and 100% at site J) than most of the single chambered tanks. A multi-compartmented tank, installed at laboratory site M, also produced a slightly higher effluent quality than a much larger single-chambered tank, but the differences were not significant at the 95% confidence level (Table A-110).

Grease, garbage grindings and trash were problems in the aerobic units that were not preceded by primary settling. In field system H, this material passed through the aeration chamber and floated on the surface of the final clarifier. Subsequently, some of this material discharged over the effluent weir. It also frequently clogged the air lift skimmer. The continuous operation of the skimmer was changed to run intermittently to allow back flushing of the skimmer pipe. This succeeded in keeping the skimmer open. At a later date (after 28 months of operation) a septic tank was placed on line at this site. This completely eliminated the problem of floating debris and grease on the clarifier.

In system G, the original effluent pump clogged twice with toilet paper, resulting in the unit filling and overflowing. After this pump failed and was replaced by one with a large intake opening, clogging never reoccurred. Problems of this type were avoided in system C, which was preceded by a septic tank. The inclusion of a septic tank or trash trap ahead of the aeration chamber seems to be good practice and is recommended in all installations utilizing aerobic treatment.

Outlet design--The design of the treatment unit outlet seems to have received little thought by many manufacturers. Weir lengths were often less than 150 mm (6 in) and in some cases the outlet was simply a 100 mm (4 in) diameter pipe. Surge flows from washing machines and baths can create flows as high as 27 L/min (7 gpm) from treatment units. In both systems C and H, floating solids were often carried into the effluent because of extremely high flow rates over relatively short weirs. The effect of surge flows on effluent quality was also noted by Bennett and Linstedt (1975). Longer weir lengths and sludge deflection baffles should be included as part of outlet design. The use of gas deflection baffles appears to be a simple way of keeping floating solids away from the weir area.

Positive solids return--Floating sludge was a problem noted with almost every aerobic treatment unit. It was caused by the production of gas bubbles by denitrification. Its impact on effluent quality can be lessened by either better outlet design as previously discussed or by providing for positive solids return.

In system C, floating sludge produced a scum blanket over 0.31 m (1 ft) thick on the clarifier. The settled sludge was returned by means of a draft tube operating off the mechanical mixer used for aeration. The aerator-mixer was run intermittently. By increasing the length of time the aerator operated, better sludge return (likely due to less denitrification) was achieved. However, there was no means of returning any sludge which did float. In system H, an air lift pump continuously returned the settled sludge and the skimmer, when working properly, returned most of the solids that did rise, keeping the surface of the clarifier clear. The batch unit in system G avoided this problem completely by the nature of its operation, ie., any floating sludge which did exist would not enter the effluent since the effluent pumped out below the liquid surface.

At laboratory site M, a good example of the need for positive solids removal was demonstrated by the rotating disk unit. Sludge was removed from the clarifier only periodically (once or twice per month). Deflection baffles kept floating solids away from the outlet weir. While the unit was able to produce a high effluent quality in terms of suspended solids, the BOD_5 level was higher than expected (Table A-110). The higher BOD values were likely due to resolubilization of organic matter in the sludge.

The extended aeration unit at this site also experienced problems with floating sludge and inadequate sludge return. A good mixed liquor suspended solids concentration never developed in this unit. Samples collected near the surface of the aeration chamber ranged in concentration from 50 to 400 mg/L MLSS over a 6-month period. These low mixed liquor values were due, in part, to the loss of solids in the effluent due to poor solids removal (Table A-110). Solids also accumulated within the aeration tank due to the low mixing input (power/unit volume) to the unit (Table A-108). Finally, a scum blanket about 3 cm (1 in) thick developed on the clarifier, and efforts to break it up and return the solids to the aeration chamber were unsuccessful.

Positive sludge return was an important feature in the design of the rotating disks and extended aeration units at laboratory site N. The entire contents of the rotating disk unit clarifier were emptied into the septic tank once per day. While this did not completely eliminate the presence of floating sludge, the effluent suspended solids were kept at a low level. In addition to being equipped with settling plates, the extended aeration unit employed an air lift to return solids to the aeration chamber two times per day for positive solids removal and return. Floating sludge was not returned by this unit, however, and neither were solids adhering to the plates. During the regular service time, these solids were easily removed. Prior to the problems of sludge build-up in the clarifier, the BOD_5 and suspended solids in the effluent averaged 13 and 25 mg/L respectively, indicating good treatment. After this problem started, the average effluent quality increased to 33 mg/L for BOD_5 and 81 mg/L for suspended solids (based on log-normal distribution).

Oxygenation and mixing--The aeration of aerobic suspended growth systems accomplishes two important functions, oxygen transfer to the bulk fluid and mixing of the biomass. Aeration is normally accomplished through the use of diffused air systems, sparged turbines or surface devices. Usually, oxygen transfer limitations are negligible because the units are so small. Analysis of oxygen transfer efficiencies of some of the laboratory units (Tables A-108 and A-109) indicate values which were low as compared to conventional municipal plant systems (Metcalf and Eddy, 1972). This is due primarily to the high power inputs to the units (constrained primarily by minimum motor sizes for these relatively small aeration tanks).

Mixing is of paramount importance in suspended growth systems. Rule of thumb requirements for mixing in aeration tanks range from 0.11 to 0.23 kw/m^3 (0.5 to 1.0 $hp/1000\ ft^3$) with this value being dependent upon basin geometry, solids concentration and the mixing device. In the units tested power inputs ranged from 0.04 to 0.83 kw/m^3 (Tables A-108 and A-109). It is apparent that the rotating disk systems provide ample mixing. The batch units, employing blowers were lower, but within the range normally acceptable for suspending solids in conventional aeration tanks. Most interesting is the low power input to the extended aeration unit of laboratory site M. As discussed above, this unit was not able to maintain an adequate mixed liquor solids level, partially because of the absence of positive sludge return and partially because of poor circulation within the aeration tank.

In an effort to reduce power requirements, some units employed cycled aeration periods (see Tables A-108, A-109 and A-112). No dissolved oxygen (D.O.) levels were monitored at site M, but measurements at laboratory site N for the batch unit indicated that D.O. levels never fell to zero in the 4 hours of OFF time during settling. The value of cycling mixed liquor to an aerobic/anaerobic mode appears desirable for nitrogen removal. System C and the extended aeration unit at laboratory site M removed over 50% of the total nitrogen via nitrification/denitrification due to aerator cycling (Tables A-110 through A-114).

Controls and alarm systems--Many of the aerobic units were equipped with some type of alarm system to indicate breakdown of mechanical parts. They ranged in complexity from air pressure sensors (field system J and extended aeration N) to circuit breakers (extended aeration N) to motion sensors and high water alarms (rotating disks - sites I and N). Some indicated problems visually (light), while others used audible signals. Those systems with buzzers also included buzzer overrides. Not all mechanical malfunctions were detected by the alarm systems. For example, neither the broken shafts of the rotating disks (site N) nor the automatic air valve malfunction of the extended aeration unit (site N) were detected. Alarm systems do not directly detect biological upsets, unless they happen to cause a failure of some mechanical component which could trigger the alarm.

In the laboratory facilities, malfunctions were detected almost immediately because of frequent inspection. The field units were visited semimonthly, so malfunctions were also detected fairly quickly. Had some of these problems occurred during normal operation, it is possible that they would not have been detected or reported by the homeowner. It is likely that there could be some indefinable time period (dependent on the frequency of routine site visits) during which an effluent of poorer quality than expected would be discharged.

Only minor problems were noted with the unit controls installed as part of this project. In an inspection of over 150 aerobic units, Voell and Vance (1974) found that during a 19 month period, over 10% of the units had problems with the timer controls. In some cases, the homeowner did not even know where the controls were located.

Sludge bulking--Sludge bulking is a phenomena noted in most suspended growth systems during at least part of their operation. Glasser (1974) attributed hydraulic and organic shock loads as being a primary reason for sludge bulking. No other study on individual home units has specifically addressed this problem. In this investigation, although qualitative shock loads (e.g. the use of toilet bowl deodorizers at site C) produced a poor settling sludge, the primary cause of filamentous bulking was found to be due to a drop in mixed liquor temperature. Glasser stated that ambient temperature variation appeared to have no adverse effect on effluent quality but no mixed liquor temperatures were presented. System H experienced several months of bulking every winter that it was monitored. Bulking would occur when the contents of the aeration chamber dropped below 15°C and normally subsided once the temperature returned to 20°C. System C also experienced this phenomena during one winter of operation. System G, a batch unit, never experienced bulking. Because of the nature of its operation, the mixed liquor temperatures were maintained about 15°C throughout most of the winter months. Sludge bulking was never a major problem in the laboratory units where the ambient air temperatures ranged from 17°C to 33°C.

It is suggested that methods be used to heat tank contents during winter months. Heaters of this type are available on the market today. One manufacturer employs a submersible aerator. The heat generated by this equipment is transferred to the mixed liquor. Another alternative is to have the aerator intake draw its air from the home through the plumbing.

Sump pump and controls--All the field installations included the construction of at least one sump (the sand filter systems normally included two). Commercial 1/4 to 1/2 hp (186 to 372 W), submersible sump pumps were purchased from local distributors and installed in the sumps. Sump pumps may or may not include level switches as part of their construction. In most cases, level sensing switches were installed separately from the pump. The majority of the sump pump systems experienced some type of problem. The most important of which was that of selecting adequate level sensing and switching equipment. Other problems included: debris being pushed into the sump during landscaping and deterioration of electrical connections within the sump. No problems were noted with the main body of the pumps (i.e. the motor and the impeller sections).

The atmosphere within the sumps proved to be a very corrosive one. Any electrical connection installed in the sump must be waterproof. If junction or control boxes are necessary, it is good practice to install them outside of the sump either in a waterproof junction box above the sump or within the home.

The selection of pump controls is critical. A good control system includes: a high water/ON sensor, a low water/OFF sensor and a high water alarm sensor. There are many different control configurations presently available for a wide range of costs. Generally, the more expensive controls are also more reliable. Of the controls installed as part of this study, the most troublesome were rubber diaphragm pressure switches. The rubber would deteriorate and allow moisture into the switch causing it to fail. Non-waterproofed controls were totally unsatisfactory. The best experience was with sealed mercury float switches. Solid-state controls appear to be the best choice for the electronics.

Maintenance--

Maintenance of units has been cited as one of the most critical problems with aerobic units (Voell and Vance, 1974; Glasser, 1974; Bennett and Linstedt, 1975; Tipton, 1975). Homeowners, in general, neither understand how their system was supposed to function nor want to be bothered with the maintenance of their system.

All treatment units, including septic tanks, require some amount of routine maintenance. There can be, however, a large variation in the frequency of this maintenance. The extended aeration unit at site N had the best defined maintenance program as set up by the manufacturer. This program included 4 scheduled visits by the manufacturer's representative during which he cleans the unit, provides proper lubrication of parts, checks components for wear and determines if the unit needs pumping (solids removal). In addition, he is required to make unscheduled visits if the homeowner notes a malfunction. Some manufacturers only provide for twice-a-year inspections of mechanical parts. In light of the large number of malfunctions encountered during this study, frequent site visits appear warranted with the more complex units. Routine maintenance falls into three main categories: solids removal, parts lubrication and replacement, and unit cleaning.

Solids removal--During the period of monitoring field systems (May 1972 to June 1974), none of the septic tanks required pumping. All except one of the aerobic units were pumped at least twice during the same period. Regular removal of excess solids is required if a higher quality effluent is to be maintained.

Several things begin to occur as solids levels increase in the aeration tank. Despite some claims by manufacturers, it is not possible to design a biological process that does not produce excess sludge. As these biological solids increase, sludge age increases and the percentage of inert solids increase. These inert solids do very little for the process. More important, however, is the increase in clarifier loading rate (expressed in terms of mass solids per surface area per time) which eventually results in solids overflow from the clarifier. (Conventional solids loading rates for extended aeration systems range from 2.4 to 6.0 kg/hr/m^2 (0.5 to 1.25 lb/ft^2/hr) based on average flow rates.) The severe hydraulic shock loads encountered by household units may have a greater impact on allowable clarifier loading than would be true of conventional systems. Normally, package plant clarifiers are generously oversized. Finally, as solids increase, sludge return capacity becomes taxed to a limit where accumulations and floating sludge may occur. Each treatment unit should be evaluated on a case by case basis for upper limits as mixed liquor solids. It appears that, dependent on the system and its use, these units should be pumped every 8 to 12 months. The pattern of growth of MLSS concentrations should be determined and when concentrations begin to reach a predetermined level, some of the sludge should be wasted. It is not necessary or desirable to pump out the entire contents of an aerobic unit (only a few hundred gallons of settled mixed liquor). Alternative methods of sludge wasting include: pumper truck, installation of an auxiliary holding tank (which would need pumping at less frequent intervals) or wasting to septic tank pretreatment.

Lubrication of parts--Most manufacturers have outlined lubrication schedules for their equipment. As far as could be determined, these schedules are adequate to ensure continued operation of moving parts.

Unit cleaning--Floating debris and trash were noted in most clarifiers. These materials should be removed during routine inspections. To this end, the surface must be readily accessible but it must also be secured for safety.

Solids also tended to adhere to clarifier surfaces. If these solids were not removed periodically, they would undergo denitrification and float to the surface. The clarifier walls can be cleaned either with a brush or, if the clarifier can be emptied, by washing down with water.

Effluent Quality--

The performance of various treatment units in terms of effluent quality has been assessed by many researchers. Some of the results of this investigation are discussed in light of other studies.

Septic tanks--Data on the quality of septic tank effluent as determined by several investigators are presented in Table A-115. As expected for this type of treatment process, effluent quality was poor with respect to organic matter, indicator organisms and the nutrients, nitrogen and phosphorus. The BOD_5 values compare reasonably well with a range of average values reported [from 93 mg/L (Thomas and Bendixen, 1969) to 240 mg/L (Bernhart, 1967)]. Average suspended solids concentrations reported in the literature ranged from 45 mg/L (Thomas and Bendixen, 1969) to 155 mg/L (Weibel, et al., 1949).

In comparing the results of various research studies, it appears that the suspended solids level in the effluent was related to (1) size of tank, (2) degree of compartmentalization, and (3) frequency of pumping. The largest tanks reported in Table A-115 were installed as part of this study and at the University of Connecticut (Laak, 1973). These studies also demonstrated the lowest levels of suspended solids. The beneficial effect of compartmentalization has been adequately demonstrated by others (Ludwig, 1950; Weibel, 1955), as well as in this study. As solids build up within a septic tank, the chances

TABLE A-115. SEPTIC TANK EFFLUENT QUALITY - INVESTIGATOR COMPARISON

Site, Dates & Source / Parameter and Statistics	This Study	Cincinnati	New York	Toronto	Cincinnati	Connecticut
	1972-1976	1947-1948	1955	1961-1966	1969	1972
	7 sites	10 tanks	19 sites	4 sites	1 site	1 tank
	(Table A-113)	(Weibel, 1949)	(Salvato, 1955)	(Bernhart, 1967)	(Thomas & Bendixen, 1969)	(Laak, 1973)
BOD_5, mg/L						
Mean (# of Samples)	138(150)	138(44)*	140(51)	240(21)†	93	120(50)
Coeff. of Variation	0.42	0.35		0.06		0.48
95% Conf. Int.	129-147	-		210-280		100-130
Range	7-480	64-256		70-385		30-280
COD, mg/L						
Mean (# of Samples)	327(152)				220	200(50)
Coeff. of Variation	0.33					0.41
95% Conf. Int.	310-344					175-225
Range	25-780					71-360
Suspended Solids, mg/L						
Mean (# of Samples)	49(148)†	155(55)*	101(51)	95(18)†	45	39(47)
Coeff. of Variation	0.16	0.54		0.08		0.98
95% Conf. Int.	44-54	-		79-115		28-50
Range	10-695	43-485		48-340		8-270
Total Nitrogen, mg/L						
Mean (# of Samples)	45(99)		36(51)		33	
Coeff. of Variation	0.40					
95% Conf. Int.	41-49					
Range	9-125					
Volume, m^3						
Mean (# of Samples)	4.7(7)	2.6(10)	-	≈ 1.8	not reported	4.0(1)
Range	3.5-7.6	1.7-7.8				

* Calculated from the average values from 10 tanks, 6 series of tests.

† Calculated on the basis of a log-normal distribution of data.

for solids washout increase. In studies where sludge build-up was accelerated (Weibel, et al., 1949; 1950), a deterioration of effluent quality occurred once the solids holding capacity had been exceeded. None of the septic tanks in this study were monitored long enough to note this phenomenon.

Aerobic units--Table A-116 presents data from several investigators on the quality of effluent from various aerobic units which have been installed at individual homes. Results of tests by the National Sanitation Foundation (NSF) are also presented for comparison. In addition, Table A-117 compares two selected manufacturers' units tested in several different studies. It is immediately apparent that there is a wide range of reported values. Average BOD_5 values ranged from 13 mg/L (McClelland, 1976) to 150 mg/L (Bennett and Linstedt, 1975) and suspended solids ranged from 17 mg/L to 150 mg/L. This variation is not surprising when considering the many factors which can adversely affect aerobic processes. Shock loads, sludge bulking, homeowner abuse or neglect and mechanical malfunctions are often cited as reasons for deteriorating effluent quality.

The aerobic processes did produce effluent of higher average quality than septic tanks, as would be expected, but periodic upsets in the aerobic units resulted in a higher variability in effluent quality than the septic tanks. Most significant to note is the effluent suspended solids concentrations which were generally as high for aerobic units as for the septic tanks (Tables A-113 through A-116). Notable exceptions of this study were the field installed aerobic units at sites H (March 1975 to May 1976) and J (Table A-114) both of which were preceded by septic tanks. In addition, the flow rate at J was unusually low. During the laboratory site N studies, the rotating disks and submerged media system also produced effluent suspended solids significantly lower than the two septic tanks (Table A-117).

All aerobic systems produced a high degree of nitrification throughout the year with the exceptions of the rotating disk unit (site N) which experienced a number of malfunctions (pump failure and clogging), resulting in periods when effluent was being discharged from the septic tank installed as part of the unit and system H which experienced a biological upset late in the monitoring period (Tables A-110, A-111 and A-114). As noted previously, denitrification was achieved in system C and in the extended aeration plant at laboratory site N as well as in the submerged media unit.

Phosphorus concentrations in the effluents from both the aerobic and anaerobic systems are variable depending primarily upon homeowner habits and uses. Most effluent phosphorus from both types of units is in the "ortho" form.

TABLE A-116. AEROBIC UNIT EFFLUENT QUALITY - INVESTIGATOR COMPARISON

Site, Dates & Source Parameter and Statistics	This Study 1972-1976 4 Sites (Table) (A-114)	Toronto 1965-1967 5 Sites (Bernhart) (1967)	New York 1972-1973 146 Sites (Voell) (1974)	Colorado - - (Bennett) (1975)	Colorado 1971-1974 55 Sites (Tipton) (1975)	Maryland 1973-1974 5 Sites (Glasser) (1975)	NSF 1971-1974 8 Units (McClelland) (1976)
BOD_5, mg/L							
Mean (# of Samples)	37(112)	47(86)*	92(146)	150(16)	144(393)	36(124)*	13†
Coeff. of Variation	0.78	0.13	-	≈0.8	-	0.21	≈0.3
95% Conf. Int.	32-42	42-52	-	-	-	31-41	-
Range	0-208	10-280	-	≈10-300	10-824	3-170	-
COD, mg/L							
Mean (# of Samples)	108(116)				385(108)		
Coeff. of Variation	0.40				-		
95% Conf. Int.	100-116				-		
Range	20-349				88-1030		
Suspended Solids, mg/L							
Mean (# of Samples)	39(117)*	94(74)*	94(146)	150(16)	122(251)	57(132)*	17†
Coeff. of Variation	0.23	0.13	-	≈0.6	-	0.22	≈0.2
95% Conf. Int.	33-46	82-108	-	-	-	49-66	-
Range	3-252	18-692	-	≈50-300	17-768	4-366	-
Number of Unit Types	4	4	3	-	6	5	8

* Calculated on the basis of a log-normal distribution of data.

† Determined graphically.

TABLE A-117. SUMMARY OF EFFLUENT DATA OF VARIOUS AEROBIC TREATMENT UNITS BY UNIT TYPE

Parameter and Statistics / Site, Dates & Source	Extended Aeration (Manufacturer #1) Bernhart 1967 1 unit	Tipton 1975 1 unit	Patterson 1977 42 sites	Glasser 1974 1 unit	NSF 1972 1 unit	This Study (lab unit) site M
BOD_5 (unfiltered), mg/L						
Mean (# of Samples)	64(14)*	33(2)	81(42)	34(29)*	19†	53(27)*
Coeff. of Variation	0.06			0.24	0.13	0.17
95% Conf. Int.	55-74			24-47		40-69
Range	37-83	22-44		4-130		14-300
Suspended Solids, mg/L						
Mean (# of Samples)	103(14)*	41(2)	89(42)	63(29)	25†	41(27)*
Coeff. of Variation	0.07			0.19	0.25	0.20
95% Conf. Int.	84-126			47-86		32-55
Range	51-168	32-50		8-262		6-180
Total Solids, mg/L						
Mean (# of Samples)			1510(42)			

	Extended Aeration (Manufacturer #2) Patterson 1977 1 site	Glasser 1974	NSF 1972 1 unit	This Study (field unit) site J
BOD_5 (unfiltered), mg/L				
Mean (# of Samples)	24	63(20)*	28(127)*	6(11)
Coeff. of Variation		0.12	0.19	0.89
95% Conf. Int.		50-80	25-32	2-10
Range		25-115	9-156	2-18

(continued)

TABLE A-117 (continued)

Parameter and Statistics / Site, Dates & Source	Extended Aeration (Manufacturer #2) Patterson 1977 1 site	Glasser 1974	NSF 1972 1 unit	This Study (field unit) site J
Suspended Solids, mg/L				
Mean (# of Samples)	93	87(28)	24(131)*	12(11)
Coeff. of Variation		0.19	0.25	0.68
95% Conf. Int.		61-122	21-27	6-17
Range		14-366	2-412	3-31
Total Solids, mg/L				
Mean (# of Samples)	776			1210(11)
Coeff. of Variation				0.28
95% Conf. Int.				990-1440
Range				920-1920

	Batch Aeration Glasser 1974	NSF 1972	This Study (field unit) site G	This Study (lab unit) site M	This Study (lab unit) site N
BOD_5 (unfiltered), mg/L					
Mean (# of Samples)	20(21)*	<21(114)*	55(19)	17(29)*	20(65)*
Coeff. of Variation	0.28	>0.21	0.84	0.31	0.25
95% Conf. Int.	14-30	18.8-24	33-78	12-24	17-24
Range	5-67	<10-110	5-208	3-120	5-150
Suspended Solids, mg/L					
Mean (# of Samples)	40(21)*	20(116)*	65(20)	18(32)*	26(69)*
Coeff. of Variation	0.24	0.25	0.17	0.21	0.20
95% Conf. Int.	27-61.	17-22	47-90	15-23	22-30
Range	4-222	2-136	21-252	4-44	7-120

* Calculated assuming log normal data distribution.

† Calculated graphically.

INTERMITTENT SAND FILTER STUDIES

Intermittent sand filtration of septic tank and treatment unit effluents was investigated from 1973 through 1976. The main objectives of the sand filter studies were to determine the effect of applied wastewater quality, media size and hydraulic loading rate on sand filter effluent quality, length of filter runs, filter clogging and the amount and type of maintenance required.

Laboratory Studies

The laboratory experiments were performed at laboratory site N and involved the application of septic tank and aerobic unit effluent to 24-10.2 cm (4 in) sand columns (Stothoff, 1976). Three types of sand, two hydraulic loading rates and two effluent types were employed in the study. The effective size and uniformity coefficient of the sands are listed below:

Effective Size (mm)	Uniformity Coefficient
0.19 - 0.22	3.3 - 4.0
0.43 - 0.45	3.0 - 3.3
0.65	1.4

The hydraulic loading rates used in the study were 0.2 m/day (5 gal/day/ft^2) and 0.4 m/day (10 gal/day/ft^2). The wastewater applied to the columns was effluent from the 3780 L (1000 gal) septic tank and the 1810 L (480 gal) extended aeration aerobic unit.

The sand columns contained 76.2 cm (30 in) of sand underlain by 15.2 cm (6 in) of coarse gravel. A freeboard space of 45.7 cm (18 in) existed above the sand to allow intermittent ponding of wastewater above the sand. The columns were dosed an average of 6 times per day.

Infiltration measurements were made shortly after the columns were started up. Initially, when no ponding occurred, infiltration rates were measured by determining the time for water above the sand to fall 2.5 cm (1 in) from 3.8 to 1.3 cm (1.5 to 0.5 in). Once intermittent ponding developed (approximately 2 months) infiltration rates were measured by determining the time for water above the sand to fall 2.5 cm (1 in) . Ultimate infiltration rates were measured by collecting the entire amount of effluent flowing from the column over a specified period of time. This assumed a steady state condition of flow.

Operation and Maintenance--

Aerobic unit/sand columns--The sand columns loaded with aerobic unit effluent at an approximate rate of 0.2 m/day (5 gal/day/ft^2) had filter runs of 170-187 days for the 0.22 mm size sand, 186-275 day filter runs for 0.43 mm size sand and a 357 plus days for 0.65 mm size sand. This verified the fact that, within this range of loading rates, longer filter runs could be attained by using larger effective size sand. Failure was caused by a crust which formed on top of the sand during the filter run. The crust appeared to be an accumulation of suspended solids forming on the top of the sand forming a gelatinous crust that acted as a straining layer itself. The rate of formation of the crust was dependent upon the concentration of suspended solids in the effluent and the rate of application to the filter (Stothoff, 1976).

As shown in Figure A-44, infiltration rates through the sand columns decreased rapidly during the initial 30 to 60 days of operation. After the initial crust had formed, however, intermittent to continuous ponding occurred while infiltration rates ranged from 0.8 m/day (20 gal/day/ft^2) to a low of 0.14 m/day (3.5 gal/day/ft^2).

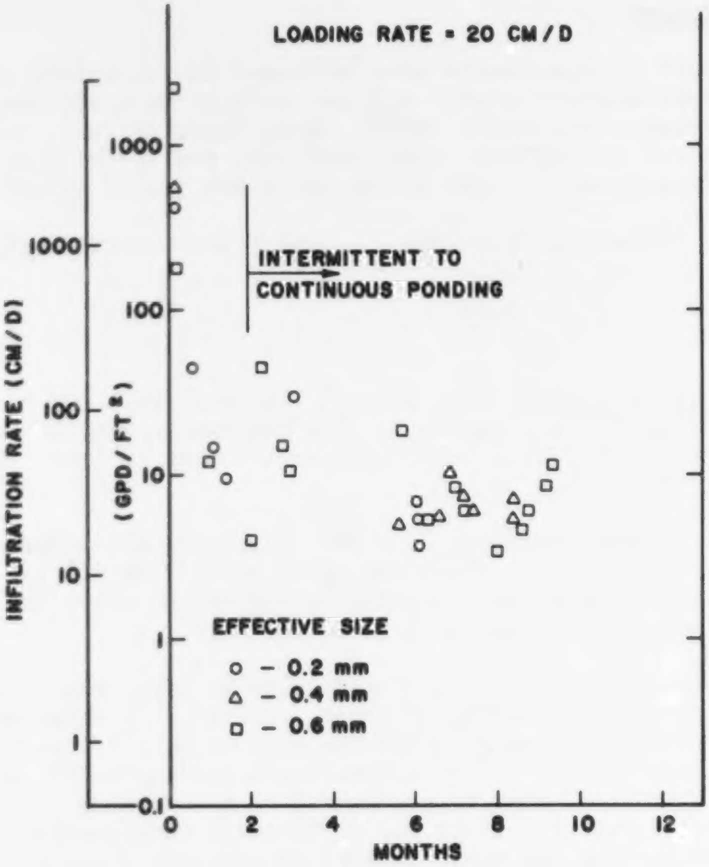


Figure A-44. Infiltration rate decline of sands loaded with aerobic unit effluent.

Although the average loading rate was 20 cm/day (5 gal/day/ft^2) failure [defined as 45 cm (18 in) of ponding for 24 hours] occurred when instantaneous loading rates became more than twice the average loading rate. These periodic high loading rates increased the ponding height and subsequently increased flow rates (up to values as high as 40 cm/day (10 gal/day/ft^2). At failure, however, the increase in flow rate for the 0.22 and 0.43 mm sand was not great enough to assimilate the periodic high loading rate. Only the 0.65 mm sand columns operated for 357 days without failing.

After evaluating a number of maintenance techniques, the recommended method for aerobic effluent was the removal of the top crust of suspended solids (0.5 to 1.5 cm) and immediate wastewater reloading. Infiltration capacity of the sand increased to 0.6 to 2.0 m/day (15-50 gal/day/ft^2) after this maintenance was performed.

The aerobic unit-sand columns loaded at approximately 0.4 m/day (10 gal/day/ft^2) produced filter runs ranging from 1 to 167 days. The high variability in filter run lengths was due to the high hydraulic loading rates. Infiltration rates through these sand columns decreased rapidly over the first 48 days of operation, dropping to a range of 1.6 to 0.4 m/day (40 to 10 gal/day/ft^2). The lowest measured infiltration rate was .084 m/day (2.1 gal/day/ft^2) and only a slight increase in infiltration rate occurred as the height of ponding increased. Run lengths were not significantly different for the different sand sizes and any maintenance other than removing the top crust (1-2 cm) resulted in exceedingly short filter runs.

Septic tanks/sand columns--The sand columns loaded with septic tank effluent at an approximate rate of 0.2 m/day (5 gal/day/ft^2) had filter runs of 6-314 days for the 0.22 mm size sand, 212-283 day filter runs for 0.43 mm size sand and a 523 day filter run for 0.65 mm size sand. Thus, within this range of loading rates, longer filter runs were attained by using larger effective size sand. Failure was caused by an accumulation of biological material (crust) within the top 5 to 10 cm (2 to 4 in) of the sand.

As shown in Figure A-45, a logarithmic decline in infiltration rate occurred during failure leading to continuous ponding of the sand and an ultimate infiltration rate as low as .012 m/day (0.30 gal/day/ft^2). There was an initial time period after the start-up of the sand columns when there was a very gradual decline in infiltration rate. The length of this time period largely influenced the lengths of the filter runs.

The height of ponding above the sand also had an effect on the infiltration rate of the sand. Instantaneous flow rates ranging from .16 to .34 m/day (4 to 8.5 gal/day/ft^2) were recorded when 46 cm (18 in) of wastewater had ponded above the sand. As the ponding height decreased, the infiltration rates decreased. At 0-5.1 cm (0 to 2 in) of ponding, infiltration rates dropped to a range of .01 to .03 m/day (0.26 gal/day/ft^2 to 0.8 gal/day/ft^2). Sixty days of continuous wastewater ponding showed that the ponding height had a less significant effect on infiltration rate for septic tank effluents than for aerobic effluents. The septic tank effluent infiltration rates were considerably lower, i.e. 0.011 to 0.022 m/day (0.28 to 0.55 gal/day/ft^2), than those recorded when aerobic unit effluent was applied to the sand columns.

Various maintenance techniques were attempted to regenerate the failed sand columns. As shown in Figure A-45, the removal of the top 5 cm of sand, raking the sand surface, or short periods of resting (5-10 days) did not effectively regenerate the filter in that subsequent filter runs were very short. Replacing or raking the top 5 to 10 cm (2 to 4 in) of sand along with

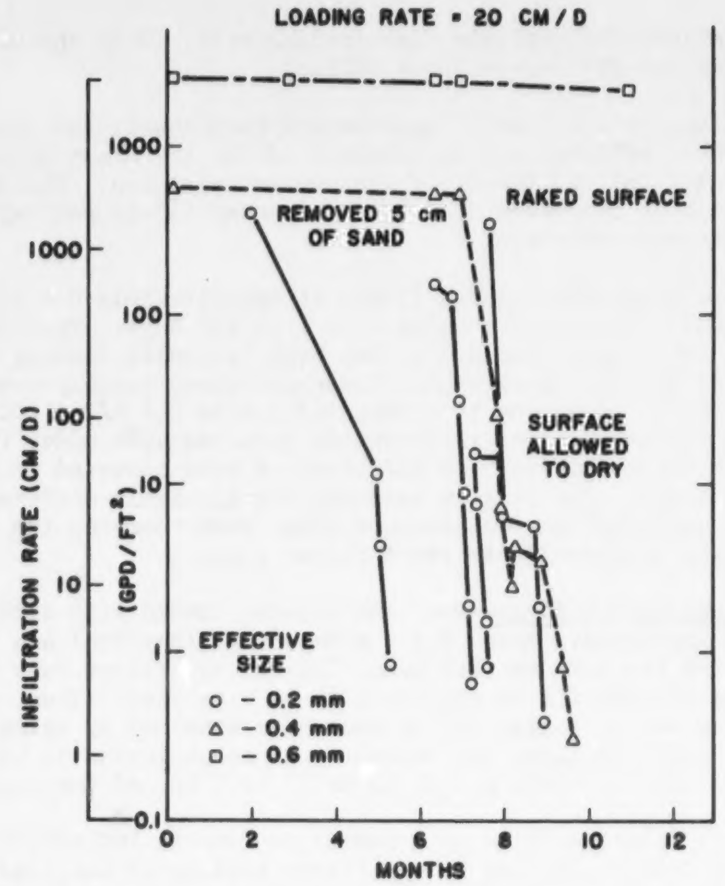


Figure A-45. Infiltration rate decline of sands loaded with septic tank effluent.

resting periods of 30 to 60 days were required to prevent filter runs shorter than 60 days. All of the maintenance techniques attempted that were not coupled with resting periods resulted in unpredictable and short filter runs.

The septic tank-sand columns loaded at approximately 0.4 m/day (10 gal/day/ft^2) produced filter runs ranging from 1 to 137 days for the 0.22 mm size sand. Filter run lengths up to 208 days were experienced by 0.43 mm and 0.65 mm sand columns.

A significant finding from the laboratory column study was that radically different clogging mechanisms occurred in the sand columns loaded with septic tank effluent as compared with those fed aerobic unit effluent. To help further understand this relationship, tensiometers were placed in each sand column at depths of 5 cm, 15 cm and 38 cm. Under saturated conditions the sand pores were filled with water, resulting in no capillary action and, therefore, zero moisture tension. As the sand drains, moisture tension increases, the larger sand pores empty and the smaller sand pores continue to hold water.

Initial wastewater loading of the sand resulted in a rapid wetting and drying cycle after each dose as shown in Figure A-46. This cycling effect decreased with depth due to dispersion of the wetting front in its downward movement. The smallest sand size (0.22 mm) exhibited the greatest tension fluctuation while all three sands reached approximately the same wetness.

The effect of increase in crust resistance, or clogging, of the aerobic unit-sand columns is depicted in Figure A-47. The accumulation of suspended solids on top of the sand surface rapidly reduced the infiltration rate. After only 13 days of operation, approximately 30 to 45 minutes was required for a dose to infiltrate into the sand. After 72 days of operation, approximate steady state moisture tensions conditions occurred between 30 minutes and two hours during which the sand remained ponded. Permanent ponding of the sand column, however, did not occur for another five weeks.

The septic tank-sand columns underwent a different type of clogging which did not significantly reduce infiltration rates, until permanent ponding (anaerobic conditions) occurred. Figure A-48 shows that after 37 days of operation a single dose required less than 30 minutes to infiltrate into the sand. This column, however, was permanently ponded less than two weeks after this reading.

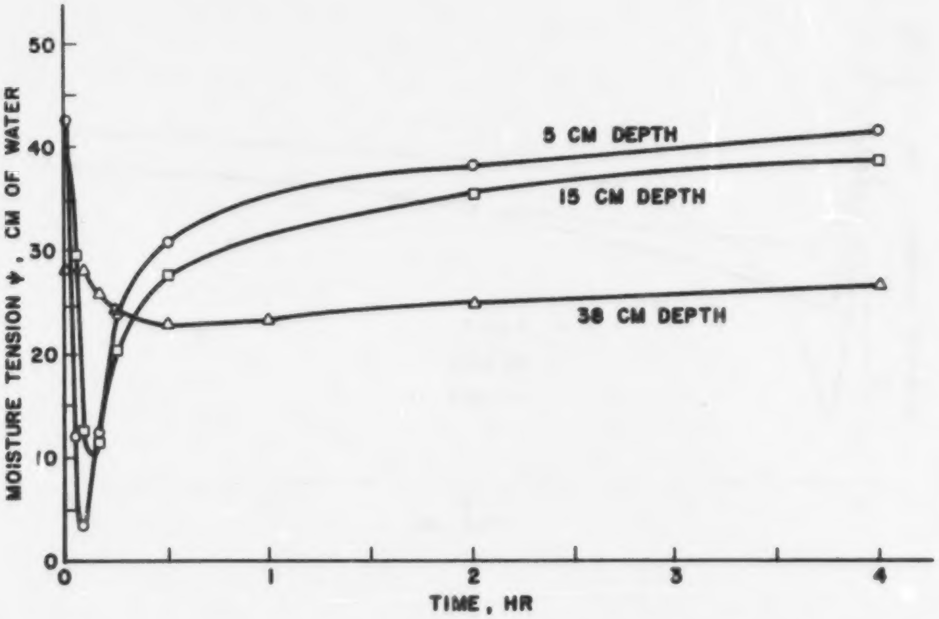


Figure A-46. Tension fluctuations with depth.

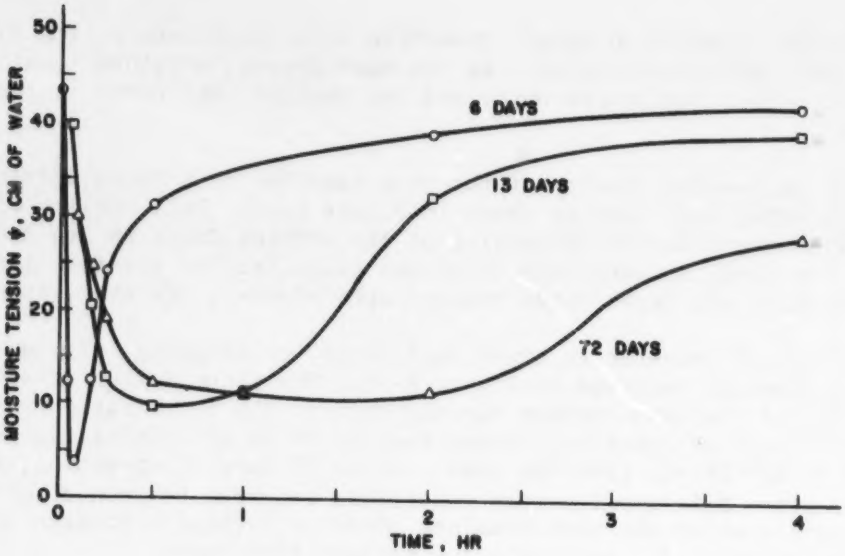


Figure A-47. Relative tension fluctuations as filter run progressed in aerobic column.

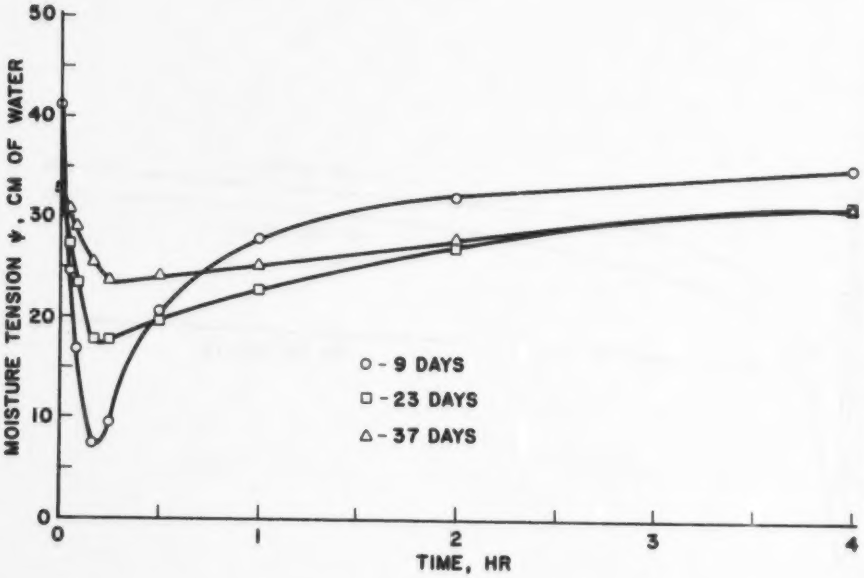


Figure A-48. Relative tension fluctuations as filter run progressed in septic column.

Effluent Quality—

The effluent quality of the aerobic unit, septic tank and the 24 sand columns was monitored from June, 1975 through December, 1975. Statistical analysis of the effluent quality data for septic tank and aerobic unit and sand filter effluents is depicted in Table A-118. Grouped values for the sand columns loaded with aerobic unit effluent and the sand columns loaded with septic tank effluent are given instead of the values recorded by each sand column. This was done for clarity and because the three-way analysis of variance showed that the applied wastewater had the most significant effect on column effluent quality. The differences in effluent quality as a function of sand sizes and loading rates were not as significant (Stothoff, 1976).

The mean BOD_5 and TSS concentrations of the aerobic unit effluent were 34 mg/L and 85 mg/L respectively. The high TSS concentration was due to the numerous biological upsets experienced by the aerobic unit. The mean BOD_5 and TSS concentrations of the septic tank effluent were 53 mg/L and 25 mg/L respectively.

The sand columns treating septic tank effluent experienced a three week maturation period during which the sand became biologically mature. This was characterized by a constantly dropping BOD_5 of the sand filter effluent and also by the increase in nitrification in the effluent. This maturation period was not experienced by the sand columns treating aerobic unit effluent.

The sand column mean effluent BOD_5 values were very low, 0.8 mg/L and 3.0 mg/L for the aerobic and septic tank columns respectively. The TSS concentrations for the aerobic and septic columns were 8 mg/L and 19 mg/L respectively. This represented an 89% TSS reduction of the aerobic unit effluent and only a 24 percent TSS reduction of the septic tank effluent. Total nitrogen concentrations for the aerobic and septic sand columns were 35.9 mg/L and 42.1 mg/L respectively. Almost all of the nitrogen was in the NO_3-N form in the effluents. Although nitrification did occur, little nitrogen was actually removed in the columns.

Two to four log reductions in fecal coliform concentrations were observed in all columns. The septic tank-sand columns also showed some reduction in *Pseudomonas aeruginosa*, while the aerobic unit-sand columns did not.

Field Installations

The field sand filter experiments were performed at field sites E, H and J. Septic tank effluent was applied to sand filters at site E while aerobic unit effluent was applied to sand filters at site H (Sauer, 1975; Sauer, et al., 1976). At site J, septic tank effluent was applied to one sand filter while aerobic unit effluent was applied to a second sand filter. All field sand filters ranged from 1.3 to 1.5 m^2 (14 to 16 ft^2) in area and contained 61 to 76 cm (24 to 30 in) of washed sand. The sand was underlain by 15 cm (6 in) of pea gravel and 15 cm (6 in) of coarse gravel. The effective size and uniformity coefficient of the sand used at each site were as follows:

TABLE A-118. SAND FILTER EFFLUENT QUALITY - LABORATORY STUDIES - LAB SITE N

Parameter and Statistics	(June 1975 - December 1975) Septic Tank Pretreatment: Septic Tank Effluent	Septic Tank Pretreatment: Sand Filter Effluent	Aerobic Unit Pretreatment: Extended Aeration Effluent	Aerobic Unit Pretreatment: Sand Filter Effluent
BOD_5, mg/L				
Mean (# of Samples)	53(31)	3.0(12)	34(30)	0.8(12)
Coeff. of Variation	0.26	0.7	0.72	1.03
95% Conf. Int.	48-59	0.8-5.1	24-44	0.2-1.5
Range	25-91	0-10	6-170	0-4
Suspended Solids, mg/L				
Mean (# of Samples)	25(31)	19(12)	85(30)	8(12)
Coeff. of Variation	0.50	0.62	1.46	1.26
95% Conf. Int.	20-29	9-30	60-110	3-19
Range	10-74	0-54	20-840	0-70
Volatile Suspended, mg/L				
Mean (# of Samples)	18(31)	5(12)	59(30)	3(12)
Coeff. of Variation	0.47	0.95	0.74	1.49
95% Conf. Int.	15-21	1-9	43-75	1-6
Range	5-42	0-26	6-170	0-25
Total Phosphorus, mg-P/L				
Mean (# of Samples)	38(22)	25	34(22)	19
Coeff. of Variation	0.23	0.21	0.38	0.26
95% Conf. Int.	34-42	17-31	28-40	11-25
Range	27-55	11-34	17-59	10-30
Orthophosphorus, mg-P/L				
Mean (# of Samples)	29(22)	18	25(22)	13
Coeff. of Variation	0.19	0.32	0.31	0.36
95% Conf. Int.	27-32	10-26	21-28	7-20
Range	20-42	4-27	14-45	5-22

(continued)

TABLE A-118 (continued)

Parameter and Statistics	Septic Tank Effluent	Sand Filter Effluent	Extended Aeration Effluent	Sand Filter Effluent
Total Nitrogen, mg-N/L				
Mean (# of Samples)	57(22)	42	40	36
Coeff. of Variation	0.24	0.21	0.33	0.45
95% Conf. Int.	51-63	30-52	34-45	16-68
Range	40-87	22-56	17-68	14-106
Ammonia Nitrogen, mg-N/L				
Mean (# of Samples)	46(22)	0.2	0.1(22)	0.2
Coeff. of Variation	0.45	2.30	2.22	3.33
95% Conf. Int.	36-55	<0.1-1.7	<0.1-0.1	<0.1-1.9
Range	0-83	<0.1-4.0	<0.1-0.3	<0.1-4.5
Nitrate Nitrogen, mg-N/L				
Mean (# of Samples)	5.5(22)	41	34(22)	34
Coeff. of Variation	2.98	0.21	0.39	0.37
95% Conf. Int.	<0.1-13	29-52	28-40	15-47
Range	<0.1-60	21-55	10-67	14-55
Fecal Coliforms, log#/L				
Mean (# of Samples)	8.0(10)	5.6	6.2(10)	2.5
Coeff. of Variation	0.05	0.32	0.10	0.52
95% Conf. Int.	7.8-8.2	4.3-6.4	5.8-6.6	0-3.7
Range	7.5-8.5	2.6-7.0	5.5-7.1	0-4.6
Ps. aeruginosa, Log#/L				
Mean (# of Samples)	4.5	3.8	4.0	4.1
Coeff. of Variation	0.12	0.27	0.25	0.34
95% Conf. Int.	4.2-4.8	2.7-4.8	3.3-4.5	3.0-5.0
Range	3.7-5.1	2.3-5.1	3.3-5.5	0-6.2

	Effective Size (mm)	Uniformity Coefficient
Field Site E	0.43 - 0.45	3.0 - 3.3
Field Site H	0.19 - 0.22	3.3 - 4.0
Field Site J	0.28	2.8

It is important to note that the sands used at field sites E and H were also used at laboratory site N. The sand was washed pit run sand which was locally available and relatively inexpensive.

The sand filters were enclosed in concrete block basins and were placed below ground level to prevent freezing problems. The top 10 cm (4 in) of the basins were above ground level to allow an insulated and removable cover to be fastened to the top of the filter. The covers prevented the accumulation of debris on the sand surface, reduced odor, eliminated freezing problems and allowed easy access to the sand surface. An open space of approximately 40 cm (16 in) above the sand surface allowed intermittent ponding of wastewater above the sand.

The distribution system for the sand filters consisted of a 5.1 cm (2 in) plastic pipe with an up-turned elbow located in the center of the bed. A splash plate was placed underneath the outlet elbow to reduce erosion of the sand surface. The collection system at the filter bottom consisted of a 10.2 cm (4 in) perforated pipe which was vented above the sand surface. Further description of the sand filters can be found elsewhere (Sauer, 1975).

The hydraulic loading rates employed for the sand filter studies ranged from 0.08 and 1.6 m/day (2 and 40 gal/day/ft^2) with the rates primarily between 0.08 and 0.4 m/day (2 and 10 gal/day/ft^2). Excessive groundwater infiltration sometimes caused the rates to become exceedingly high. The sand filters were dosed (4 to 13 times/day) by a submersible pump with a controlled volume of wastewater. The frequency was dependent upon the amount of wastewater generated each day.

Operation and Maintenance--

Aerobic unit sand filter--Initially field sand filters were operated at a hydraulic loading rate ranging from 0.14 to .16 m/day (3.5 to 4.0 gal/day/ft^2). Filter run lengths of up to 289 days were experienced. The clogging layer formed on top of the sand surface. Failure of the sand filters was defined as the time when the ponded wastewater had reached a level 30 cm (12 in) above the sand surface. At the time of failure, the clogging mat of suspended solids had become 2-2.5 cm (3/4-1 in) in depth. Analysis of this mat showed that it contained over 70% volatile material. The sand immediately below the crusted layer was clean and contained less than 0.5% more volatile matter than clean sand. Removal of the crusted layer along with 2.5 cm (1 in) of top sand and the replacement of 2.5 cm (1 in) of clean sand was the best maintenance method.

Further experimental studies evaluated sand filter loadings of up to 0.29 m/day (7.3 gal/day/ft^2), and as low as .04 m/day (1.0 gal/day/ft^2). The low flow rate sand filter showed only a slight accumulation of suspended solids on the sand surface, whereas the high loading rate caused a much more rapid clogging mat accumulation on the sand surface. To further characterize this type of filter failure infiltration rates through the sand filter were monitored and are plotted in Figure A-49.

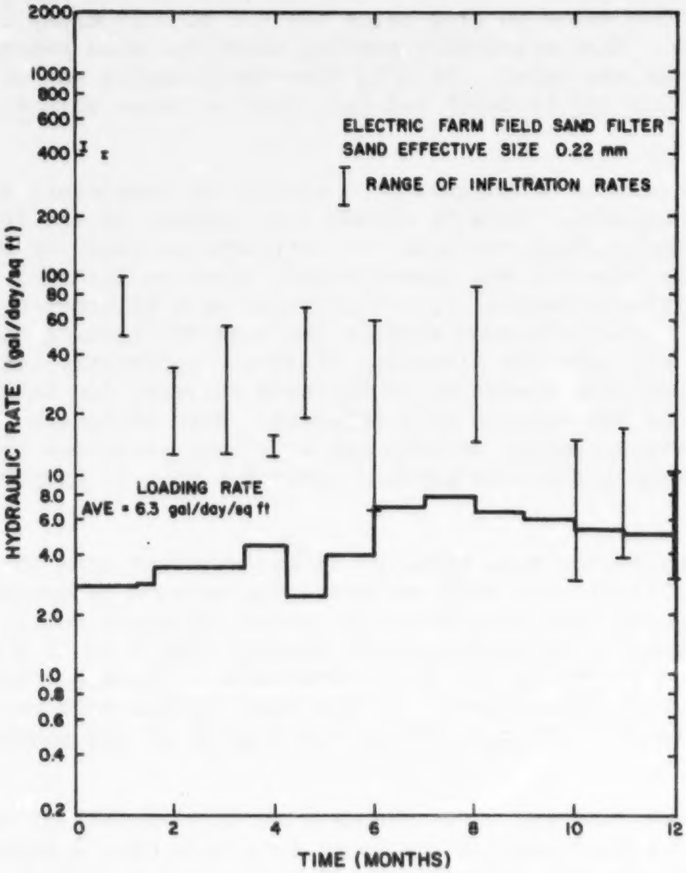


Figure A-49. Infiltration rate decline of sand loaded with aerobic unit effluent, site H.

An immediate decline in infiltration rate occurred during the first month of application. This appeared to be due to an accumulation of suspended solids, which were strained from the wastewater. This mat of accumulated solids formed on top of the sand surface and did not penetrate into the sand. The formation of the solids mat continued throughout the filter run, possibly indicating a non-degradable nature of the suspended solids in the effluent of the aerobic treatment unit. Flow rates through the crust ranged from 2-4 m/day (50 to 100 gal/day/ft^2) at this time.

During the remaining filter run, infiltration rates ranged from 0.12 to 4 m/day (3 to 100 gal/day/ft^2). The range of infiltration rates was dependent upon the amount of time the crusted material on top of the sand had remained unponded before being dosed with wastewater. When continuous ponding occurred, infiltration rates decreased to as low as 0.12 m/day (3 gal/day/ft^2); however, when the crust was allowed to dry and crack open, infiltration rates were as high as 4 m/day (100 gal/day/ft^2). Eventually, continuous ponding predominated causing infiltration rates to drop below the 0.2 to 0.24 m/day (5 to 6 gal/day/ft^2) loading rate. When wastewater ponding above the sand reached 30 cm (12 in), the filter run was ended. At this time the clogging mat on top of the sand was 3.8 cm (1.5 in) in depth and infiltration rates were $\leq$ .12 m/day (3.0 gal/day/ft^2).

Various maintenance techniques were studied to regenerate the clogged sand filters and columns. Results showed that removal of the solids mat from the top of the sand surface restored the infiltrative capacity to 50 to 100 gal/day/ft^2. This capacity was approximately equal to the sand capacity after the initial logarithmic decline in infiltration rate at start-up. This allowed succeeding filter runs to be made without the need for resting the sand bed, thus eliminating the need for alternate filters. Apparently little active biological decomposition occurs below the sand surface, due to the low soluble organic material in the aerobic unit effluent. This statement is only true when the sand surface remains aerobic and at a high oxidation reduction potential. It also assumes that the aerobic treatment unit is properly operated and maintained.

The length of filter runs appeared to be dependent upon the mass of suspended solids applied to the sand surface. The mass was determined by the hydraulic loading rate and the concentration of suspended solids in the aerobic unit effluent. At loading rates ranging from 0.16 to 0.24 m/day (4 to 6 gal/day/ft^2) and suspended solids concentration ≈ 50 mg/L, filter run lengths of one year have been experienced. At the same loading rate and suspended solids concentrations ≈ 100 mg/L filter run lengths of six months have been experienced.

Based on these findings it is recommended that removal of the crust on the sand surface be performed at six month intervals when a properly operated aerobic treatment unit and hydraulic loading rates of 0.2 m/day (5 gal/day/ft^2) are employed. This offers a factor of safety in the event of periodic upsets by the aerobic unit.

Septic tank/sand filters--Initially field sand filter systems were operated at high hydraulic loading rates (0.56 to 1.6 m/day) [14-40 gal/day/ft^2]. From these experiments it was determined that sand filter failure was quite rapid (45 to 80 days) and also that regeneration of the sand filters was best performed by removing the top 10 cm (4 in) of sand and replacing it with clean sand. In an attempt to obtain longer filter runs and improved effluent quality, the remaining field experiments were operated at approximately 0.2 m/day (5 gal/day/ft^2). Length of filter runs at this loading rate ranged from 83 to 143 days when maintenance consisting of replacing the top 10 cm (4 in) of sand was performed. The addition of short resting periods (30 to 45 days) to the sand replacement maintenance insured the longer filter runs. Another maintenance technique consisting of physically raking the sand surface along with resting the sand 30 to 60 days resulted in filter run lengths ranging from 67 to 91 days. Other maintenance techniques such as replacing or raking the sand surface without corresponding resting periods or resting periods ranging from 1 to 90 days without corresponding physical maintenance to the sand, resulted in short filter runs of less than 30 days. An important conclusion from these studies was that sand replacement or raking should be combined with resting periods of at least 30 to 60 days to insure run lengths of approximately 90 days.

All of the above experiments were performed with sands having an effective size of 0.43 mm and a uniformity coefficient from 3.0 to 3.3. Other field experiments utilizing smaller effective size sands showed similar trends of sand filter failure; however, corresponding run lengths were shorter. Summarizing the field experimental data, laboratory data and literature information, Table A-119 shows the relation between effective size of sand and filter run length.

TABLE A-119. SEPTIC TANK - SAND FILTER - RUN LENGTH VS. EFFECTIVE SIZE

Sand		Filter Run Length
Effective Size (mm)	Uniformity Coefficient	(Days)
0.2	3 - 4	30
0.4	3	90
0.6	1.4	150

These run lengths were based on a 0.2 m/day (5 gal/day/ft^2) loading rate and maintenance consisting of sand replacement or raking combined with resting periods. The failure point was defined as 30 cm (12 in) of ponding above the sand surface. Decreasing the hydraulic loading rate would yield longer filter runs; however, detailed investigations using lower loading rates were not performed in this study. It was felt that sufficient information using less than 0.2 m/day (5 gal/day/ft^2) loading rates already existed in the literature.

To help further explain and possibly predict sand filter failures, hydraulic flow rates through the sand filters were monitored. Figure A-50 shows the infiltration rate decline and the effect of maintenance on a sand filter loaded with septic tank effluent. The sand had an effective size of 0.43 mm and an initial saturated hydraulic conductivity of 23 m/day (565 gal/day/ft^2). The average hydraulic loading rate was 0.2 m/day (5 gal/day/ft^2). After an initial period of start-up, a logarithmic decline in infiltration rate occurred leading to continuous ponding of the sand and an ultimate infiltration rate for all runs between 0.02 to 0.04 m/day (0.5 and 1.0 gal/day/ft^2). The length of this initial break in time period largely influenced the lengths of filter runs.

Replacing the top 10 cm (4 in) of sand with no resting regenerated the hydraulic capacity of the filter bed to about .24 m/day (6 gal/day/ft^2). Replacing the top 10 cm (4 in) of sand along with one month of resting regenerated the hydraulic capacity to about 14.4 m/day (360 dal/day/ft^2) or 60% of the initial saturated hydraulic conductivity.

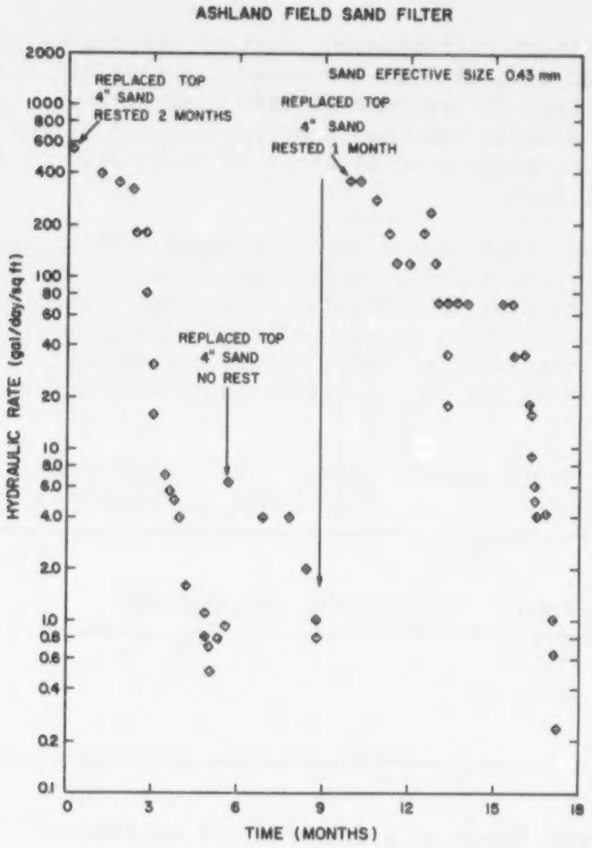


Figure A-50. Effect of maintenance on sands loaded with septic tank effluent.

Using clean or regenerated sand, initial ponding times were approximately 4 minutes. As the sand filter matured, ponding times increased to 8 to 15 minutes. A major portion of the filter run was conducted when ponding times ranged from 15 to 30 minutes and infiltration rates ranged from 1.4 to 4.5 m/day (36 to 114 gal/day/ft^2). Once ponding times increased to one hour or greater, continuous ponding occurred rapidly. This was experienced in many filter runs and can be explained by analyzing the dosing arrangement.

For design purposes, it is recommended that a hydraulic loading rate of 0.2 m/day (5 gal/day/ft^2) be used to determine the required surface area of the sand filter. It is also recommended that an additional sand filter of equal size be installed due to the dosing effect of septic tank effluent. Application of effluent onto the sand filters should alternate between the two beds, with time periods of loading and resting dependent upon the effective size of the sand, as shown in Table A-119. Maintenance of the sand filters should be performed immediately after its loading period and involves replacing the top 4 inches of sand with clean sand or raking the top 5 to 10 cm (2 to 4 in) of sand.

Effluent Quality--

Chemical and bacterial parameters--The sand filter effluent qualities from field sites E, H and J are shown in Tables A-120 and A-121 for septic tank and aerobic unit pretreatment, respectively. One of the major conclusions from the study was that the sand filter effluent quality was quite similar for the septic tank and aerobic treatment unit effluents. This was quite surprising since the organic strength of the septic tank effluent was higher than the aerobic unit effluent.

Analysis of the data listed in Tables A-120 and A-121 shows that in all the sand filter studies the mean BOD_5 concentration was ≤ 10 mg/L. Only during the initial start-up and maturation period of the sand filters were the BOD_5 concentrations significantly greater than the mean value. Throughout the filter runs, as biological and physical clogging occurred, the quality of the sand filter effluent improved. The BOD_5 from the septic tank was largely soluble and was assimilated biologically during passage through the sand bed. Conversely, the BOD_5 from the aerobic unit effluent was primarily associated with the suspended solids which were generally removed at the surface of the sand.

Mean values for the total suspended solids concentrations of the sand filter effluents were usually ≤ 15 mg/L. A significant percentage of these suspended solids were observed to be non-volatile fine sand grains which were washed from the filter bed and seen in the effluent.

Concentrations of total nitrogen through the sand filters were relatively unchanged; however, almost complete nitrification of the septic tank effluent occurred. Only after the sand surface remained continuously ponded for over 3 weeks would ammonia nitrogen appear in the sand filter effluent. Out of the usual filter run lengths of approximately 90 to 140 days, there were only from 7 to 14 days when significant ammonia nitrogen concentrations were found in the sand filter effluents.

Total and orthophosphate concentrations were initially reduced 20 to 30% by sand filters with clean sand. As the sand filters aged, however, little or no phosphorus reduction was found. Reduction of phosphorus through sand was probably due to the absorption of the PO_4-anion to organic material and sand grains.

Fecal coliforms, total coliforms and fecal streptococci counts in the septic tank effluent were reduced 2 logs by the sand filters. A 2 to 3 log reduction of Ps. aeruginosa was also experienced. These results compare to a one log reduction of fecal coliform, total coliform, fecal streptococci and Ps. aeruginosa by sand filters treating aerobic unit effluents which were loaded at a rate of .14 to .28 m/day (3.5 to 7.0 gal/day/ft^2). Reducing the loading rate to .06 m/day (1.5 gal/day/ft^2), reduced the fecal coliform and total coliform counts of the aerobic unit effluent by 2 logs. Although considerable reductions of bacteria through the sand filters did occur, the

TABLE A-120. FIELD SAND FILTER EFFLUENT QUALITY - SEPTIC TANK PRETREATMENT

Site and Dates	E				J	
	Aug 1974 - Mar 1975		Mar 1975 - Dec 1976		June 1976 - Dec 1976	
Parameter and Statistics	Septic Tank Effluent	Sand Filter Effluent	Septic Tank Effluent	Sand Filter Effluent	Septic Tank Effluent	Sand Filter Effluent
Filter Loading m/d (gal/d/ft^2)	.19-.21 (4.8-5.3)		.09-.34 (2.2-8.4)		.11-.16 (2.8-3.9)	
BOD$_5$ (unfiltered), mg/L						
Mean (# of Samples)	123(13)	9(25)*	109(13)	7.1(21)*	57(12)	1.6(24)*
Coeff. of Variation	0.56	0.48	0.54	0.44	0.27	0.89
95% Conf. Int.	81-165	6-14	73-145	4.8-10	47-66	1.3-1.9
Range	22-235	1-87	7-215	1-42	40-85	1-3
BOD$_5$ (filtered), mg/L						
Mean (# of Samples)	87(13)	6.5(25)*	69(7)		42(3)	
Coeff. of Variation	0.64	0.53	0.64		0.18	
95% Conf. Int.	53-121	4.3-9.8	16-122		23-61	
Range	10-175	1-49	2-170		35-50	
COD, mg/L						
Mean (# of Samples)	280(10)	52(19)*	271(13)	62(18)*	208(12)	46(24)*
Coeff. of Variation	0.54	0.17	0.48	0.08	0.28	0.12
95% Conf. Int.	173-387	42-65	192-351	53-73	172-244	38-56
Range	65-520	22-175	25-460	36-110	157-350	20-135
Total Solids, mg/L						
Mean (# of Samples)			1060(13)	1160(21)	1340(12)	1380(24)
Coeff. of Variation			0.36	0.34	0.37	0.31
95% Conf. Int.			830-1290	980-1340	1030-1650	1200-1560
Range			410-1800	310-1900	920-2310	910-2340

(continued)

TABLE A-120 (continued)

Parameter and Statistics / Site and Dates	E - Aug 1974-Mar 1975 Septic Tank Effluent	E - Aug 1974-Mar 1975 Sand Filter Effluent	E - Mar 1975-Dec 1976 Septic Tank Effluent	E - Mar 1975-Dec 1976 Sand Filter Effluent	J - June 1976-Dec 1976 Septic Tank Effluent	J - June 1976-Dec 1976 Sand Filter Effluent
Volatile Solids, mg/L						
Mean (# of Samples)			250(13)	340(21)	260(12)	245(24)
Coeff. of Variation			0.41	0.54	0.34	0.38
95% Conf. Int.			190-310	260-420	210-310	210-280
Range			70-420	110-690	120-410	120-440
Suspended Solids, mg/L						
Mean (# of Samples)	48(11)*	7(23)*	39(13)*	2.9(21)*	34(12)*	14(24)*
Coeff. of Variation	0.24	0.68	0.18	1.61	0.07	0.24
95% Conf. Int.	26-89	3.9-12	26-57	1.3-6.3	29-40	11-18
Range	10-256	1-400	14-135	1-28	23-50	5-35
Volatile Suspended, mg/L						
Mean (# of Samples)	37(11)*	5(24)*	28(13)*	1.4(21)*	28(12)*	5.0(24)*
Coeff. of Variation	0.22	0.73	0.24	4.67	0.08	0.33
95% Conf. Int.	21-63	3-8.2	17-45	0.7-2.8	23-33	4.0-6.2
Range	7-124	1-100	5-105	0-14	17-48	1-13
Total Nitrogen, mg-N/L						
Mean (# of Samples)	26(9)	24(19)*	27(9)	33(14)	36(12)	
Coeff. of Variation	0.42	0.21	0.84	0.68	0.07	
95% Conf. Int.	17-34	17-33	10-45	20-46	34-37	
Range	16-51	12-193	9-86	9-99	31-40	
Ammonia Nitrogen, mg-N/L						
Mean (# of Samples)	21(11)	0.9(19)*	19(9)	5.5(15)	26(12)	<0.1
Coeff. of Variation	0.38	-	0.79	1.04	0.22	-
95% Conf. Int.	15-26	-	7-30	2.4-8.6	22-29	-
Range	11-35	<0.1-16	7-56	0-17	10-31	<0.1-0.5

(continued)

TABLE A-120 (continued)

Parameter and Statistics / Site and Dates	E - Aug 1974-Mar 1975		E - Mar 1975-Dec 1976		J - June 1976-Dec 1976	
	Septic Tank Effluent	Sand Filter Effluent	Septic Tank Effluent	Sand Filter Effluent	Septic Tank Effluent	Sand Filter Effluent
Nitrate Nitrogen, mg-N/L						
Mean (# of Samples)	0.3(11)	20(19)*	0.1(9)	32(16)	0.2(12)	34(24)
Coeff. of Variation	-	0.48	0.7	1.04	1.21	0.19
95% Conf. Int.	-	10-40	<0.1-0.2	14-50	0-0.3	31-37
Range	-	0.3-172	<0.1-0.4	3-117	0.1-0.8	29-55
Total Phosphorus, mg-P/L						
Mean (# of Samples)	11(11)	9(19)*	12(10)	16.5(16)	16(10)	15(20)
Coeff. of Variation	0.43	0.21	0.48	0.46	0.08	0.11
95% Conf. Int.	8-14	7-11	8-16	12-21	15-17	14-16
Range	6-22	5-27	4-24	6-30	15-18	11-19
Orthophosphorus, mg-P/L						
Mean (# of Samples)	10(11)	7(19)*	8(10)	10(16)	15(12)	13(24)
Coeff. of Variation	0.46	0.27	0.61	0.43	0.13	0.10
95% Conf. Int.	7-13	5.4-9.0	5-12	7.8-12	13-16	12-14
Range	4-19	2-19	3-20	5-20	10-17	11-16
Total Coliforms, log#/L						
Mean (# of Samples)	7.0(12)	3.9(25)	6.4(10)	4.0(18)	7.0(14)	4.4(29)
Coeff. of Variation	0.08	1.08	0.19	0.63	0.08	0.14
95% Conf. Int.	6.7-7.2	2.2-5.6	5.7-7.2	2.7-5.2	6.7-7.3	4.2-4.6
Range	6.3-7.4	1.3-6.9	4.8-7.5	2.3-6.5	6.3-7.9	3.0-5.2
Fecal Coliforms, log#/L						
Mean (# of Samples)	6.8(14)	3.9(27)	6.3(10)	3.9(19)	6.8(14)	4.2(28)
Coeff. of Variation	0.09	1.51	0.24	0.38	0.07	0.18
95% Conf. Int.	6.6-7.0	1.6-6.2	5.4-7.2	3.2-4.6	6.6-7.0	3.9-4.5
Range	6.1-7.3	1.0-5.6	4.0-7.6	2.5-6.5	6.2-7.4	2.7-5.2

(continued)

TABLE A-120 (continued)

Parameter and Statistics / Site and Dates	E - Aug 1974-Mar 1975		E - Mar 1975-Dec 1976		J - June 1976-Dec 1976	
	Septic Tank Effluent	Sand Filter Effluent	Septic Tank Effluent	Sand Filter Effluent	Septic Tank Effluent	Sand Filter Effluent
Fecal Strep, log#/L						
Mean (# of Samples	4.5(14)	2.4(28)	5.0(10)	2.4(19)	5.2(14)	2.9(28)
Coeff. of Variation	0.40	3.0	0.36	0.68	0.20	0.45
95% Conf. Int.	4.1-4.8	1.0-5.2	4.0-6.0	1.6-3.2	4.8-5.7	2.4-3.4
Range	3.0-5.3	1.3-5.7	3.9-7.6	1.0-3.8	4.0-6.4	1.3-4.5
Ps. aeruginosa, log#/L						
Mean (# of Samples)			5.1(8)	2.4(16)	3.9(8)	2.8(16)
Coeff. of Variation			0.35	0.63	0.14	0.31
95% Conf. Int.			3.9-6.3	1.6-3.2	3.5-4.2	2.3-3.3
Range			3.3-7.4	0.3-3.9	3.3-4.4	2.2-4.4

* Calculated on the basis of log-normal distribution of data.

TABLE A-121. SAND FILTER EFFLUENT QUALITY - FIELD SITES WITH AEROBIC UNIT PRETREATMENT

Site and Dates	H				J	
	Aug 1973 - Feb 1975		Mar 1975 - May 1976		June 1976 - Dec 1976	
Parameter and Statistics	Aerobic Unit Effluent	Sand Filter Effluent	Aerobic Unit Effluent	Sand Filter Effluent	Aerobic Unit Effluent	Sand Filter Effluent
Filter Loading m/d (gal/d/ft^2)	.11-.16 (2.7-4.0)		.04-2.9 (1-7.3)		.11-.16 (2.8-3.9)	
BOD$_5$ (unfiltered), mg/L						
Mean (# of Samples)	26(48)	3.5(56)*	48(15)	3.5(24)*	6(11)	1.2(22)*
Coeff. of Variation	0.20	0.73	0.67	0.92	0.89	1.62
95% Conf. Int.	21-31	2.7-4.5	30-65	2.2-5.7	2-10	1.0-1.4
Range	6-86	1-56	10-145	1-39	2-18	1-3
BOD$_5$ (filtered), mg/L						
Mean (# of Samples)	9(45)	2.8(56)*	22(14)			
Coeff. of Variation	0.41	0.81	1.35			
95% Conf. Int.	7-12	2.2-3.5	5-39			
Range	1-60	1-30	3-120			
COD, mg/L						
Mean (# of Samples)	82(50)	27(57)*	89(15)	26(23)*	71(11)	40(23)*
Coeff. of Variation	0.12	0.28	0.51	0.20	0.51	0.12
95% Conf. Int.	70-95	21-35	66-120	20-34	47-95	33-48
Range	20-210	4-285	28-220	11-140	30-160	12-96
Total Solids, mg/L						
Mean (# of Samples)			720(15)	720(24)	1210(11)	1240(23)
Coeff. of Variation			0.08	0.17	0.28	0.30
95% Conf. Int.			690-750	670-770	990-1440	1080-1400
Range			600-800	530-1200	920-1920	860-2400

(continued)

TABLE A-121 (continued)

Site and Dates Parameter and Statistics	H - Aug 1973-Feb 1975		H - Mar 1975-May 1976		J - June 1976-Dec 1976	
	Aerobic Unit Effluent	Sand Filter Effluent	Aerobic Unit Effluent	Sand Filter Effluent	Aerobic Unit Effluent	Sand Filter Effluent
Volatile Solids, mg/L						
Mean (# of Samples)			230(15)	210(24)	210(11)	210(23)
Coeff. of Variation			0.29	0.44	0.41	0.42
95% Conf. Int.			190-270	170-250	150-270	180-250
Range			130-320	40-440	80-300	20-430
Suspended Solids, mg/L						
Mean (# of Samples)	48(51)*	11(58)*	23(15)*	6.4(23)*	12(11)	3.9(20)*
Coeff. of Variation	0.18	0.41	0.20	0.88	0.68	0.50
95% Conf. Int.	39-58	8.5-14	16-32	3.2-13	6-17	2.8-5.4
Range	5-164	1-100	8-45	1-40	3-31	1-14
Volatile Suspended, mg/L						
Mean (# of Samples)	33(51)*	5.3(58)*	15(15)*	2.8(23)*	5(11)	2.0(20)*
Coeff. of Variation	0.22	0.46	0.33	1.25	0.61	1.23
95% Conf. Int.	27-41	4.3-6.5	9-24	1.6-4.9	3-7	1.3-3.0
Range	4-136	<1-42	2-40	1-18	1-10	<1-8
Total Nitrogen, mg-N/L						
Mean (# of Samples)	39(46)	36(51)	31(8)	29(10)	31(12)	
Coeff. of Variation	0.33	0.37	0.29	0.40	0.08	
95% Conf. Int.	35-43	32-40	24-39	21-37	30-32	
Range	20-78	9-70	15-44	6-46	26-34	
Ammonia Nitrogen, mg-N/L						
Mean (# of Samples)	0.4(47)	0.3(51)	7(8)	0.5(10)	<0.1(12)	<0.1
Coeff. of Variation	3.65	-	1.16	1.75	-	-
95% Conf. Int.	<0.1-1.6	-	0.2-14	<0.1-1.1	-	-
Range	<0.1-19	<0.1-3	<0.1-23	<0.1-3	-	<0.1-0.7

(continued)

TABLE A-121 (continued)

Parameter and Statistics / Site and Dates	H - Aug 1973-Feb 1975 Aerobic Unit Effluent	H - Aug 1973-Feb 1975 Sand Filter Effluent	H - Mar 1975-May 1976 Aerobic Unit Effluent	H - Mar 1975-May 1976 Sand Filter Effluent	J - June 1976-Dec 1976 Aerobic Unit Effluent	J - June 1976-Dec 1976 Sand Filter Effluent
Nitrate Nitrogen, mg-N/L						
Mean (# of Samples)	34(50)	35(51)	22(8)	28(10)	30(12)	30(24)
Coeff. of Variation	0.41	0.36	0.71	0.44	0.07	0.12
95% Conf. Int.	30-38	31-39	9-34	19-37	30-31	29-32
Range	0.3-72	5-68	1-39	2-46	26-33	24-46
Total Phosphorus, mg-P/L						
Mean (# of Samples)	32(41)	21(49)	17(8)	17(10)	9(10)	8(20)
Coeff. of Variation	0.73	0.59	0.22	0.55	0.19	0.31
95% Conf. Int.	24-39	17-26	14-20	11-24	8-10	6.4-8.6
Range	9-140	6-46	13-24	7-34	6-12	5-17
Orthophosphorus, mg-P/L						
Mean (# of Samples)	28(41)	21(49)	10(8)	9(10)	8(12)	7(24)
Coeff. of Variation	0.54	0.60	0.31	0.64	0.16	0.15
95% Conf. Int.	23-33	17-25	8-13	5-13	7-9	6.2-7.0
Range	9-51	5-46	7-16	3-25	6-10	4-8
Total Coliforms, log#/L						
Mean (# of Samples)		5.2(52)	6.2(9)	4.4(15)	4.4(14)	2.4(28)
Coeff. of Variation		0.32	0.22	0.27	0.12	0.47
95% Conf. Int.		4.7-5.7	5.3-7.1	3.7-5.0	4.2-4.6	2.0-2.8
Range		3.7-7.1	4.0-7.7	3.3-6.7	3.8-5.3	1.0-3.7
Fecal Coliforms, log#/L						
Mean (# of Samples)	5.3(51)	4.0(56)	5.2(9)	3.9(15)	4.1(14)	1.8(28)
Coeff. of Variation	0.24	0.61	0.36	0.45	0.16	0.70
95% Conf. Int.	5.1-5.4	3.3-4.6	4.0-6.4	2.9-4.8	3.8-4.4	1.3-2.2
Range	4.2-6.4	3.0-5.6	2.8-7.3	3.9-6.6	3.3-5.2	1.0-2.8

(continued)

TABLE A-121 (continued)

Parameter and Statistics / Site and Dates	H - Aug 1973-Feb 1975		H - Mar 1975-May 1976		J - June 1976-Dec 1976	
	Aerobic Unit Effluent	Sand Filter Effluent	Aerobic Unit Effluent	Sand Filter Effluent	Aerobic Unit Effluent	Sand Filter Effluent
Fecal Strep, log#/L						
Mean (# of Samples)	4.4(52)	3.2(58)	4.4(9)	3.2(15)	3.4(14)	2.0(28)
Coeff. of Variation	0.53	3.69	0.58	0.90	0.34	0.95
95% Conf. Int.	4.2-4.6	-	2.9-5.9	1.6-4.7	2.9-3.8	1.2-2.7
Range	2.3-6.1	1.5-5.1	2.0-6.3	1.0-5.0	2.1-4.4	1.0-3.3
Ps. aeruginosa, log#/L						
Mean (# of Samples)			4.0(8)	2.9(13)	5.0(8)	2.8(16)
Coeff. of Variation			0.23	0.29	0.19	0.21
95% Conf. Int.			3.4-4.5	2.4-3.4	4.3-5.6	2.5-3.1
Range			2.9-5.0	2.3-4.4	3.9-6.4	2.3-3.4

*Calculated on the basis of a log-normal distribution of data.

effluent levels of coliforms remained higher than current federal surface discharge recommendations for total and fecal coliforms.

Virus retention in intermittent sand filtration--A column containing 60 cm of conditioned sand was dosed with 25 cm of septic tank effluent containing $\approx 10^7$ PFU/mL of Po-1 (see Appendix C). Dosing continued at the same daily rate thereafter, but no virus was added to the septic tank effluent. The titers of the daily sand column effluent samples ranged downward from 2.5×10^3 PFU/mL on day 1 to no virus detected at a 10^{-1} dilution of column effluent on day 7. Clearly, the retentive capacity of the sand had been exceeded by once-a-day dosing with 25 cm of septic tank effluent. This single dose was approximately equal to the void volume of the sand column. Virus retention was approximately 99.96%.

The question was examined further using the 0.09 m^2 (1 ft^2) surface area, 76 cm (30 in deep sand filters at laboratory site N. These were dosed with an average of 14 and 28.5 cm/day, divided into approximately seven randomly distributed equal doses per day, i.e., one filter was receiving ≈ 2 cm per dose, an average of seven times per day. Each filter was inoculated with 5×10^9 PFU of Po-1. If this had been uniformly distributed in the first day's septic tank effluent throughout, the virus titers for the column effluents should have been 2.5×10^6 and 1.2×10^6 PFU/mL, respectively, in the absence of any virus retention. Effluents collected daily from each column for a week and tested in tissue cultures after only a 2-fold dilution revealed no virus penetration.

By the time the experiment was to be repeated, the more heavily loaded of the two filters had crusted. The top several inches of sand were removed, and the effluents from this filter remained turbid throughout the rest of the experiment. More virus (1.3×10^{10} PFU of Po-1) was inoculated into these same two filters and into another which had been receiving ≈ 29 cm of grey water per day. No virus was detected in the effluents from the filters receiving grey water or the smaller daily volume of septic tank effluent, but the turbid effluent from the filter dosed with 28.5 cm septic tank effluent per day was positive at a 10^{-2} dilution on day 1 and a 10^{-1} dilution on day 6 (other samples were negative). This indicates that even the reconditioned filter reduced the virus content of the septic tank effluent by more than 5 $\log_{10}$. Based on these laboratory studies one might conclude that the key to successful sand filtration of septic tank effluent, in terms of virus retention, is to divide the fluid load into enough small portions so as to permit increased retention time in the filter to allow adhesion of virus to physical and biological surfaces within the filter.

NUTRIENT REMOVAL

Nitrogen

Denitrification Systems--

Application of the technology of denitrification to small scale wastewater systems introduces a host of potential problems not present with large-scale systems. The main problems are economics and maintenance requirements. An additional potential problem is the health hazard associated with the availability of methanol for inadvertant human consumption (Posner, 1975).

The requirements for a small scale system are that it be of a simple design, economical to construct and maintain, have an adequate service life, and be effective in removing nitrate without adding other pollutants. The system would be placed down stream from the effluent disposal field so that the nitrified effluent could flow through the denitrification system, hopefully by gravity. Alternatively, the system could be designed to accept effluent from nitrification reactors such as a sand filter. Because nitrification will also oxidize essentially all of the endogenous organic carbon, an exogenous energy source is required.

Alternate energy sources--The first problem in designing a denitrification system to be placed under seepage beds is choosing the energy source. One approach would be to use an easily obtainable, slowly decomposable, solid energy source with sufficient material being available for the planned life of the system (e.g., 10 years might be an economical life time). The release of energy material ideally would be enough for denitrification but would not add excess C to the effluent. The other approach is to supply the energy in a readily available form such as methanol (CH_3OH). The latter would have the disadvantage that the CH_3OH must be metered with the influent at the required rate. This would require periodic maintenance. Even distribution of the liquid energy source for proper mixing would pose another problem. The solid energy material has the distinct disadvantage that it is difficult to envisage a material capable of sustaining denitrification for such long periods. Also, should the system malfunction, corrective action would require extensive renovation. Therefore, research was necessary to determine the method which will be best suited for a home sewage disposal system.

Several energy sources were evaluated in this study. These included (a) peat from a marsh at the west end of Lake Wingra (Madison, Wisconsin); (b) coniferous forest litter from the University of Wisconsin Arboretum, Madison; (c) paper mill sludges; (d) oat straw; (e) molasses and (f) methanol. All dry materials were ground to <2 mm particle size prior to use.

The denitrification rates (Table A-122) are based on 100 mg soluble C added and varying incubation times due to the rates of various energy sources to reduce nitrate a significant amount, e.g., 4 days for peat and 0.75 days for molasses (procedures are detailed elsewhere, Sikora & Keeney, 1976). Methanol and the paper mill sludges gave rapid denitrification rates, while with straw, forest litter and peat, denitrification was slower. The molasses value was misleading because far more molasses was added than necessary, resulting in an unrealistically low rate. A rate similar to CH_3OH would be expected because molasses should be a readily available energy source. The data show that the sludges were far superior to the other solid energy sources. Sources which yield rapid rates would be preferable because the size of the bed or reactor needed for denitrification should be minimized. Inasmuch as these data (Table A-122) were expressed on the basis of soluble C which would generally be the energy source available for denitrification, the variations observed in rates must be for other reasons. One reason, i.e. molasses, was that more soluble C was present than necessary for reduction. Another reason may be the availability of the soluble C present. Peat gave a low denitrifi-

TABLE A-122. ALTERNATE ENERGY SOURCE DATA

Energy Material	Denitrifier (MPN per bottle)	Soluble C (mg per bottle)	Denitrification Rate (g NO_3^--N removed/d /100 mg sol. C)
Peat	2.4×10^7	84	72
Coniferous forest litter	6.7×10^6	63	471
Kimberly Paper Mill sludge	1.7×10^8	14	7140
Lakeview Paper Mill sludge	2.5×10^5	17	7820
Oat straw	2.4×10^9	122	763
Sand molasses	1.3×10^6	930	30
Sand methanol	4.9×10^6	2	16250

cation rate even though sufficient C was available. Other constituents such as clays and sulfur found in the paper mill sludges may affect the rate. The number of organisms present was another variable. Therefore, several singular aspect explanations can be given to partially explain the data. However, in designing a long-term denitrification system, the interaction of all factors, as well as the rate of release and immobilization of C, must be evaluated.

Continuous flow columns dosed with methanol--Plexiglas columns (10.2 cm I.D. x 64 cm) were filled with 1 cm diameter $CaCO_3$ chips and sealed. Sampling ports and Pt black electrodes were placed at 14, 34, and 54 cm (Figure A-51). Residence times were determined by chloride analysis. Column flow rates were controlled by peristaltic pumps. Methanol addition to the influent was also controlled by the pump and mixing was accomplished by using a mixing coil. Gas samples were collected by water displacement. Prior to sampling the gas collection bottle and the syringe were flushed several times with helium and samples were taken through the rubber septum by first injecting a volume of helium equal to the sampling volume (0.5 mL). Investigations were performed at 5°C, 13°C, and 20°C.

A ratio of 1.4 CH_3OH-C/NO_3^--N was used. The influent used was nitrified (aerated for an extended period) septic tank effluent diluted 1:36 with tap water to ensure low endogenous carbon and NO_3^- was added as KNO_3 to equal a 40 or 80 mg N/L concentration. The NO_3-N and BOD_5 levels of mechanically aerated septic tank effluent averaged 37 mg/L and 36 mg/L, respectively. Three ports were used for sampling in the 20°C experiment while for the 5°C and 13°C experiments, four sampling ports were used.

Figure A-52 indicates the general curvilinear relationship found at all three temperatures between the decrease in NO_3^- and time in the column experiments. The data in Figure A-52 are from the 5°C study. Nitrite was monitored intermittently throughout the study and the maximum NO_2^--N recorded was about

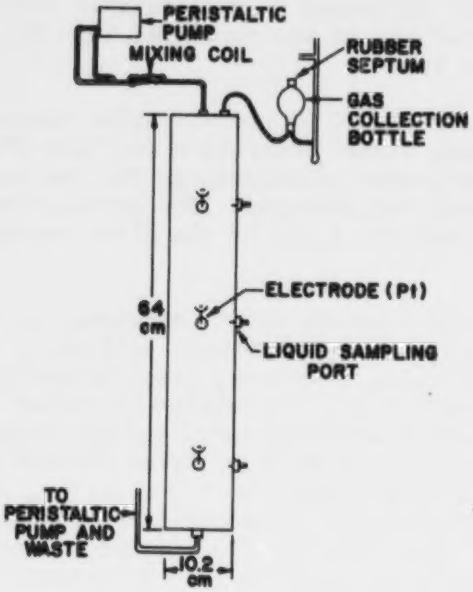

PERISTALTIC PUMP
MIXING COIL
RUBBER SEPTUM
GAS COLLECTION BOTTLE
64 cm
ELECTRODE (Pt)
LIQUID SAMPLING PORT
TO PERISTALTIC PUMP AND WASTE
10.2 cm

Figure A-51. Diagram of continuous flow column apparatus.

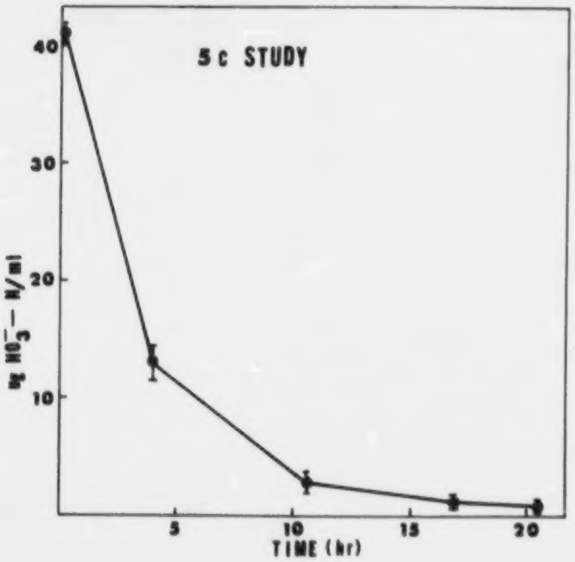


Figure A-52. Nitrate-N decrease with time in continuous flow columns at 5°C.

2.0 mg/L. Nitrite always decreased to undetectable levels as denitrification neared completion. Therefore, it was concluded that NO_2^- accumulation would not be a problem in this system.

Plots of the logarithm of NO_3^--N concentration versus time were made and tested for linearity using linear regression analysis (Figures A-53 through A-55). Significant first-order relationships for the rate of NO_3^--N reduction were obtained at all three temperatures. The correlation coefficients were highly significant (1%) and the slope of the lines represent first-order constants (k).

Suspended solids or biological growth measurements were not performed in these column studies because of the inherent difficulty of accurately measuring biomass on the limestone chips. However, batch anaerobic (helium) incubation studies were performed with flasks containing nitrified septic tank effluent and CH_3OH. Biomass, C and N were monitored during incubation. A yield coefficient of approximately 0.8 mg of cells/mg CH_3OH-C oxidized was obtained. This yield coefficient is lower than the 1.1 g cells/g CH_3OH reported by Snedecor and Cooney (1974) with a thermophilic mixed culture under aerobic incubation conditions. Schroeder and Busch (1968) reported 0.9 to 1.0 g cells/g glucose-C for cells grown using nitrate as a terminal electron acceptor. Cell yield is an important parameter for consideration in that clogging is a major drawback with packed bed denitrification systems.

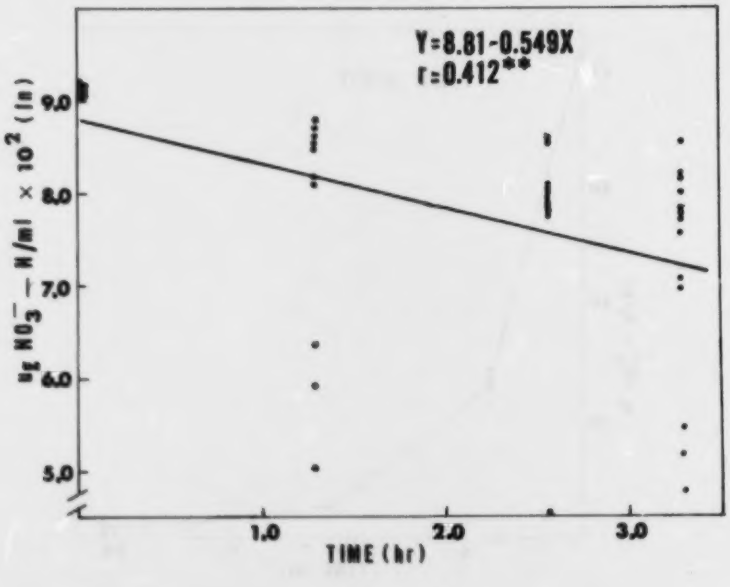


Figure A-53. Linear regression analysis of NO_3^--N decrease (ln) with time in continuous flow columns at 20°C.

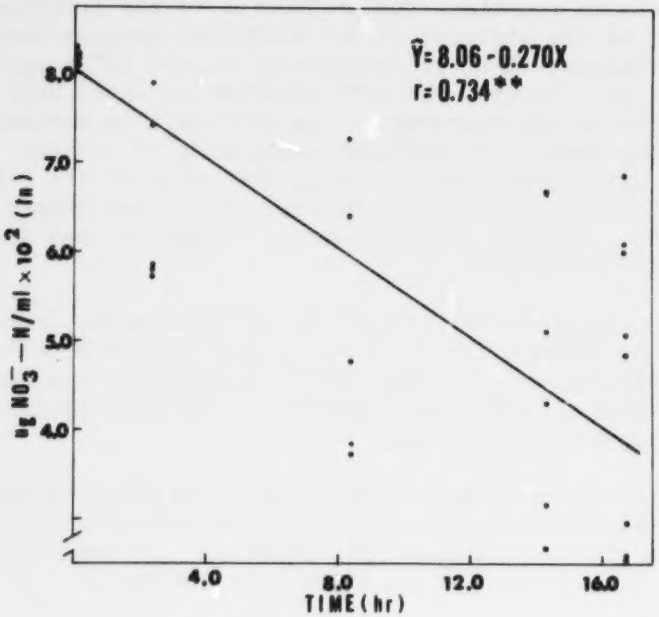


Figure A-54. Linear regression analysis of NO_3^--N decrease (ln) with time in continuous flow columns at 13°C.

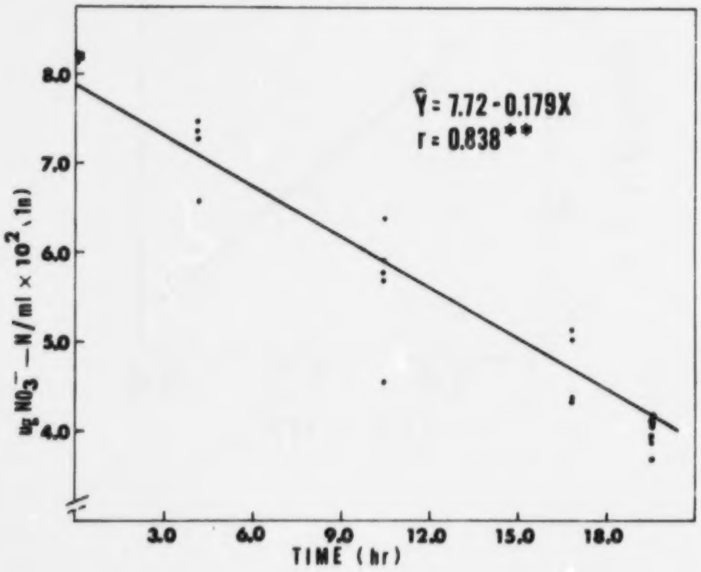


Figure A-55. Linear regression analysis of NO_3^--N decrease (ln) with time in continuous flow columns at 5°C.

Use of regression equations for prediction of the residence time necessary for zero NO_3^- in the effluent at the different temperatures results in the following estimates: 5°C, approximately 17 hrs; 13°C, approximately 13 hrs; and, 20°C, 1 to 2 hours. The 20°C equation is based upon an 80 mg N/L influent concentration and therefore it is difficult to accurately predict the denitrification rate of an influent containing 40 mg N/L. Since this reaction gives a first-order relationship, the rates at the higher initial concentrations would be more rapid. However, in a preliminary experiment it was found that at 20°C, complete denitrification occurs in less than 2 hrs for influent containing 40 mg N/L.

Therefore, it appeared that prediction of NO_3^- removal in nitrified septic tank effluent using first-order k values was suitable in a packed bed denitrification system similar to that described. However, it was noted that, with time, slightly higher k values were obtained. This was probably due to increasing biomass within the columns.

The temperature effect was estimated by using the k values obtained from the first-order relationships and graphical analysis (Figure A-56) indicates that these values fit an Arrenhius relationship. The activation energy was calculated from the graph and found to be 7.32 kcal/mole.

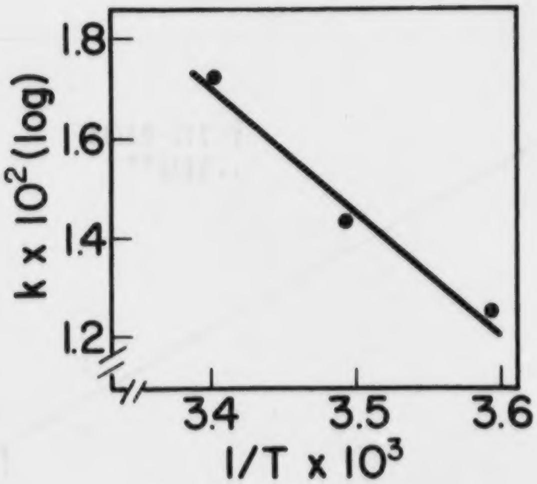


Figure A-56. Logarithm of k values versus reciprocal temperature.

At all three temperatures, nitrogen gas was detected at the highest concentrations (Table A-123). Trace amounts (<1%) of O_2 also were detected. Because of the continuous flow approach used, this O_2 may have resulted from external contamination when the influent bottles were changed and when the samples were taken. If contamination was the source, N_2 would also be a contaminant in concentrations from 1 to 5 µM (equal to atmospheric $O_2:N_2$ ratio). The remaining N_2 would be from microbial origin.

TABLE A-123. GAS CONCENTRATIONS (µ MOLES/cc GAS PRODUCED) IN COLUMN ATMOSPHERE AT THREE TEMPERATURES

Gas	Temperature (C)					
	5		13		20	
	Mean	Range	Mean	Range	Mean	Range
O_2	0.98	0.28-1.20	0.60	.30-1.20	0.19	0.02-0.30
N_2	45.50	43.20-47.20	45.71	45.00-47.00	40.00	26.02-50.50
CO_2	t*	t	t	t	0.12	t-0.6
CH_4	t	t	t	t	0.07	0.02-0.10

*t, trace.

Nitrous oxide (N_2O), a gaseous intermediate of denitrification, was detected only occasionally during the experiments. This was expected since, in a semi-closed system N_2O is rapidly transformed to the next byproduct. Its high solubility in water also makes detection difficult (Cheng and Bremner, 1965). Nitric oxide (NO), a denitrification intermediate normally associated with non-biological NO_3^- reduction and occasionally encountered during denitrification in soil, was never detected in our experiments. Measurable amounts of CO_2 were obtained only at 20°C.

One disturbing event occurred during this experiment. After approximately 2 to 4 months of continuous flow, clogging occurred due to accumulation of a crust formed at the top of each column at all temperatures. Drying the columns out in the air for one day was sufficient to re-establish the original flow rate but clogging reoccurred at much more frequent intervals. Therefore, it would appear that field systems should have some means for alternate dosing and resting of the denitrification system for a short period of time. Another critical point in designing a denitrification system would be allowance for adequate gas release. Reduction of the flow rate due to gas build-up has been noted by Requa and Schroeder (1973) in their column which was not equipped for gas release.

The oxidation-reduction potential associated with the actively denitrifying system is about +225 mv (Patrick and Mahaptra, 1968). However, with the exception of the 5°C study, the readings we obtained were generally lower than +225 mv (Table A-124). This may be because the columns had undergone continuous flow for a number of weeks before the values in the table were determined, resulting in decreased redox potentials with time, probably due to the accumulation of reduced products during prolonged decomposition. Nitrate, being in the highest concentration at the top of the columns, would cause the values to be highest in this zone with a general decrease down the columns.

TABLE A-124. OXIDATION-REDUCTION POTENTIALS (Eh_7, MV) AT THREE LEVELS IN COLUMNS AT THREE TEMPERATURES

Distance Down Column	Temperature (C)					
	5		13		20	
	Mean	Range	Mean	Range	Mean	Range
13 cm	+323	+290-+378	+206	-25-+400	+169	+118-+193
33 cm	+278	+238-+328	+63	-20-+170	+189	+78-+238
53 cm	+88	+10-+158	+134	+5-+270	+152	+93-+183

Some increase in oxidation-reduction potentials should occur as temperature decreases. According to the Nernst equation, an increase of 10 to 20 mv should occur as temperature drops from 20°C to 5°C, assuming the concentration of oxidized and reduced species do not change. Considerably larger increases in mv readings occurred in this study, especially at 5°C.

The electrode responses to nitrified and denitrified effluent were sufficiently adequate to suggest that a field system could be monitored for efficiency using Pt-black electrodes. When NO_3^- was present in the area of the columns monitored, the electrodes generally had readings of approximately +200 mv. If NO_3^- was not present, the readings would progressively decrease to zero and below. Also, when clogging of the top portion of the columns began due to buildup of biomass, the electrode in this area gave high negative values even in the presence of NO_3^-. Therefore, both denitrification efficiency and/or biomass buildup in packed columns could be monitored employing oxidation-reduction electrodes.

Results of carbon analysis for the 5°C study only are given due to technical difficulties in instrumentation which obviated the other data. However, while the relative rates of reduction and oxidation changed with temperature, the absolute values obtained would not be likely to differ greatly. Therefore, the conclusions based on C values obtained at 5°C should also apply at 13°C and 20°C.

The data show a decrease in organic C down the columns with a concomitant increase in inorganic C. The organic C was assumed to be CH_3OH since the influent was diluted (<5 mg soluble C/mL) and the amount of metabolites formed other than biomass should be minimal. Biomass C would not be detected because samples were filtered through 0.45 μ Millipore filters before analysis.

A correlation analysis (Figure A-57) was performed on the change of $(NO_3^- + NO_2^-)$-N versus organic C down the column and a significant linear relationship (5% level) was obtained. The ratio of CH_3OH-C oxidized to $(NO_3^- + NO_2^-)$-N reduced was 0.89 as calculated by the slope of the line. This ratio is near that recorded by Smith, et al. (1972), in their continuous flow packed columns. They recorded ratios of 0.88 to 0.97 with no inclusion of dissolved oxygen (D.O.) levels.

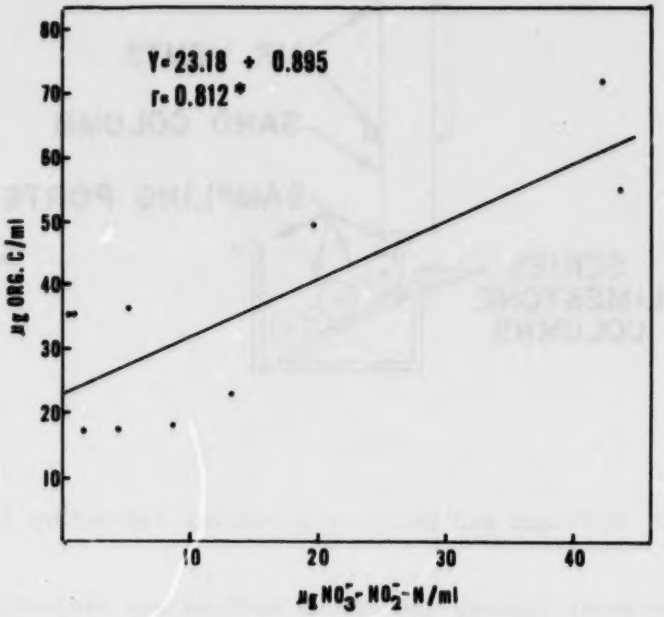


Figure A-57. Linear correlation analysis of changes in soluble organic C and NO_3^--NO_2^--N levels in continuous flow columns at 5°C.

Gravity flow system combined with phosphorus removal--The gravity flow system (Figure A-58) consisted of a 10 x 60 cm vertical Plainfield sand column followed by three 8 x 32 cm horizontal columns filled with either dolomite (0.64 or 0.96 cm diameter) or calcite (0.32 or 0.64 cm diameter). The Plainfield sand column was used to simulate the soil below the seepage bed, had air ports at 15 and 30 cm to maintain aerobic conditions, and was the site of nitrification. The limestone columns were the site of denitrification. The limestone columns contained baffles at two points for maximum exposure of effluent to limestone. The sand column was dosed once a day with approximately 500 mL of septic tank effluent (equal to about 6 cm/day) obtained from a household septic system. Fresh effluent was obtained weekly and was stored at 6°C until used. The outflow for each set of columns was adjusted so that the series of three limestone columns remained anaerobic.

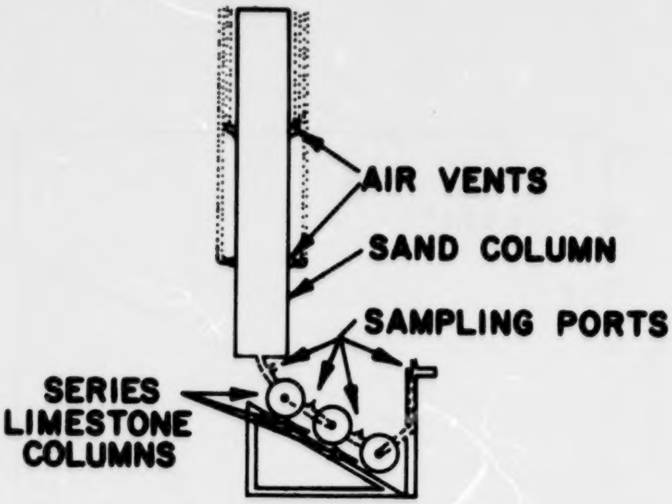


Figure A-58. Nitrogen and phosphorus removal laboratory test system.

Effluent movement through the column systems was evaluated using a chloride tracer. The residence time for the sand and limestone columns were approximately one day each. It was observed that the majority of the effluent movement through the column series occurred within one hour of the sand column dosing. Methanol was used as an energy source for denitrification and was injected daily into the second limestone column near the entrance. The injection took place about 15 minutes after the sand column was dosed so that the flow of the effluent would aid in the mixing of methanol with effluent. The methanol concentration was 1.4 CH_3OH-C/NO_3^--N, (twice the stochiometric C requirement), based on 40 mg N/L in the effluent.

With the exception of one 6°C column study, all other studies were performed in a temperature controlled room at 20°C. The sand columns were dosed separately with septic tank effluent for 4 to 6 weeks. By that time the effluent contained no NH_4^+-N, 30 to 50 mg of NO_3^--N/mL, and about 12 mg P/L. The limestone columns were then attached and their effluents were analyzed three times a week for the first month followed by once a week for the duration of the experiment. Denitrification was not substantial in the limestone columns until after 2 to 3 weeks of daily dosing. The data reported were all collected following this startup period.

The N data obtained from the four series of columns using two types and sizes of limestones indicate similar trends (Table A-125). The effluent from the sand columns (Port 1) had variable concentrations of NO_3^--N probably due to the N variability of the septic tank effluent used for dosing (Otis, et al., 1975). The occasional higher NO_3^- concentrations at Port 2 (after the first limestone column) compared to Port 1 were also considered to be the result of the initial effluent variability. On the average about 25% (ranging from 0 to 50%) of the NO_3^--N was lost during passage through the first limestone column (one day residence time) indicating that some endogenous C was available for denitrification after nitrification. Longer residence times (3 days) without adding methanol did not increase NO_3^- removal. With the addition of methanol in the second limestone column, NO_3^- declined further with from 60 to 100% loss (Port 3) depending upon the initial NO_3^- concentrations entering the limestone columns. After the third column (Port 4), the NO_3^--N was at undetectable levels in all systems.

TABLE A-125. NITRATE PLUS NITRITE CONCENTRATIONS (μg N/mL) AT PORTS IN VARIOUS LIMESTONE SERIES COLUMNS AFTER ATTACHMENT TO SAND COLUMNS

0.64 cm (0.25 in) Dolomite					0.96 cm (0.38 in) Dolomite				
	Port					Port			
Day	1*	2	3	4	Day	1*	2	3	4
21	43.1	21.4	5.4	0.0	12	20.7	36.1	1.5	0.0
65	52.7	37.6	17.3	1.5	47	44.7	35.6	7.6	0.0
84	54.4	36.6	1.5	0.0	50	67.6	45.6	14.0	0.0
108	42.0	31.7	15.3	0.0	79	37.6	41.7	7.2	0.0
122	34.2	42.1	0.0	0.0	100	39.1	36.1	13.2	0.5
177	50.8	41.2	7.5	3.0	116	48.2	42.9	17.1	0.0
0.64 cm (0.25 in) Calcite					0.32 cm (0.13 in) Calcite				
38	23.6	37.1	7.9	0.2	46	44.5	30.4	8.9	0.0
68	15.2	15.2	9.9	0.0	84	29.9	21.2	0.0	0.0
95	29.6	29.9	2.2	0.0	103	19.3	21.7	0.0	0.0
120	31.8	28.2	1.4	0.0	137	46.2	21.2	0.0	0.0
135	27.5	33.2	0.7	0.7	155	78.5	14.5	0.0	0.0
159	30.5	21.6	1.2	1.2	168	23.1	22.9	4.9	0.0
185	21.2	19.0	0.5	0.0	194	28.2	21.2	2.2	0.5

*Port 1 followed the sand column, port 2 followed the first limestone column, port 3 followed the second limestone column (methanol added), and port 4 followed the third limestone column.

In the 6°C study, NO_3^- removal was also quite high (50 to 100%) after one day residence time following methanol injection (Table A-126). It should be noted that nitrification was also complete at 6°C using the sand column with a slightly greater than one day residence time. The simplified methanol injection procedure gave rates comparable to those obtained in studies where methanol was mixed with effluent prior to entrance into a continuous flow column containing 0.96 cm (0.39 in) dolomite (Sikora and Keeney, 1975). One question concerning the feasibility of a denitrification system in home waste disposal is the mechanics of introduction of methanol or any energy source to the nitrified effluent. Data presented here indicate a single daily injection of energy material into nitrified effluent is a feasible method.

TABLE A-126. NITRATE PLUS NITRITE CONCENTRATIONS (µg N/mL) AT PORTS IN 0.64 cm (0.25 in) CALCITE SERIES COLUMN AT 6°C

	0.64 cm (0.25 in) Calcite			
	Port			
Day	1*	2	3	4
24	26.7	26.2	13.2	9.2
33	29.8	27.1	11.0	11.0
47	21.6	18.7	9.4	0.0
56	21.4	24.4	0.0	0.0
68	21.9	20.6	9.6	1.2
82	18.5	18.3	1.7	0.0

* Port 1 followed the sand column, port 2 followed the first limestone column, port 3 followed the second limestone column (methanol added), and port 4 followed the third limestone column.

Total N analysis (inorganic plus organic N) of samples from the columns was performed occassionally and the total N concentrations obtained correlated (within 10%) with the inorganic N concentrations obtained by steam distillation. Therefore, we concluded that the decreases in inorganic N recorded should have been due to denitrification.

Problems did occur in the limestone columns which are characteristic of packed bed denitrification systems (English et al., 1974). Gas bubbles which were probably N_2 built-up in the columns restricting flow. One attempt was made to put in stand pipes (glass tubing with a height greater than that at Port 4) in the second column to act as gas release ports. These were generally successful. Biomass accumulation is another related problem observed in this system, but recent techniques in biomass removal could be applied here (Brown, 1975). The dosing-resting cycle suggested for seepage beds by Bouma, et al. (1974b), could also reduce biomass accumulation.

The effects of limestone size on denitrification rates were calculated from Table A-126 using the differences in NO_3^--N concentration at ports 1 and 3. The average percent removal was 67% for 0.96 cm dolomite, 84% for 0.64 cm dolomite, 83% for 0.64 cm calcite, and 92% for 0.32 cm calcite. As Smith, et al. (1971) and others have reported, denitrification increases in packed bed systems with smaller diameter bed material due to increased surface area. No differences were detected between equal diameter dolomite and calcite systems.

Evaluation of a Thiobacillus denitrificans system--Duplicate 10 x 64 cm plexiglas columns (similar to the systems used previously) were filled with a 1:1 (w/w) mixture of dolomitic limestone (1 cm diameter) and elemental sulfur (crude, lump sulfur, with a particle size of > 2 mm) and sealed. The columns were equipped with sampling ports and Pt black oxidation-reduction electrodes at 12, 32, and 52 cm from the top of the columns. The influent used was an extended aeration treated septic tank effluent. For this study the influent was diluted 1:36 to insure low endogenous C levels and NO_3^- was added as potassium nitrate (KNO_3) to equal a final concentration of 40 mg N/L. The influent flow was controlled by a peristaltic pump and the effluent flow was adjusted using screw clamps to equal the influent flow.

Thiobacillus denitrificans ATCC 23642 was grown in the medium described in Taylor, et al (1971). For column preparation, the stock culture was inoculated into 500 mL aliquots of medium and incubated under helium at 30°C for 7 to 14 days. After incubation the medium was recycled through the sulfur-limestone columns for 3 days for establishment of the organism within the columns.

The data from the continuous flow columns were obtained after attainment of steady state conditions (defined as those times when inorganic N concentrations at the sampling ports did not differ by more than 10% over a 3-day period).

Establishment of the T. denitrificans on the sulfur within columns is a random procedure, depending upon such factors as the growth stage of the culture, concentration of organisms in the enrichment media and surface area of the S. In these studies the enrichment culture was recycled through the columns for 3 days and approximately two weeks of influent continuous flow had occurred before significant denitrification was observed. In eight weeks, denitrification in the columns reached steady-state. Figure A-59 shows the curvilinear relationship obtained between the decrease in NO_3^- and time under steady-state conditions. Essentially all of the NO_3^--N had been removed from the influent the 3.3 hr residence time. Nitrite (NO_2^-) increased and then decreased with time to a level of approximately 5 mg/L NO_2^--N. Accumulation of NO_2^- was observed in the columns using shorter residence times (< 2 hr). However, with longer residence times (>3.5 hr), NO_2^- levels decreased to near zero.

A test was made to determine if the data fit a first-order relationship. A regression analysis was performed on the logarithm of NO_3^- concentration versus residence time (from Figure A-59) and a significant linear relationship was obtained (Figure A-60). Thus these findings are similar to those observed using heterotrophic denitrification.

Oxidation-reduction (redox) potentials (Eh_7), taken during steady-state conditions ranged from +160 to +300 mv (Table A-127). The readings were highest at the top of the column and decreased with depth. The redox potential normally associated with an actively denitrifying soil system is about +225 mv (Patrick and Mahapatra, 1968) which is approximately the midpoint in the range of readings obtained.

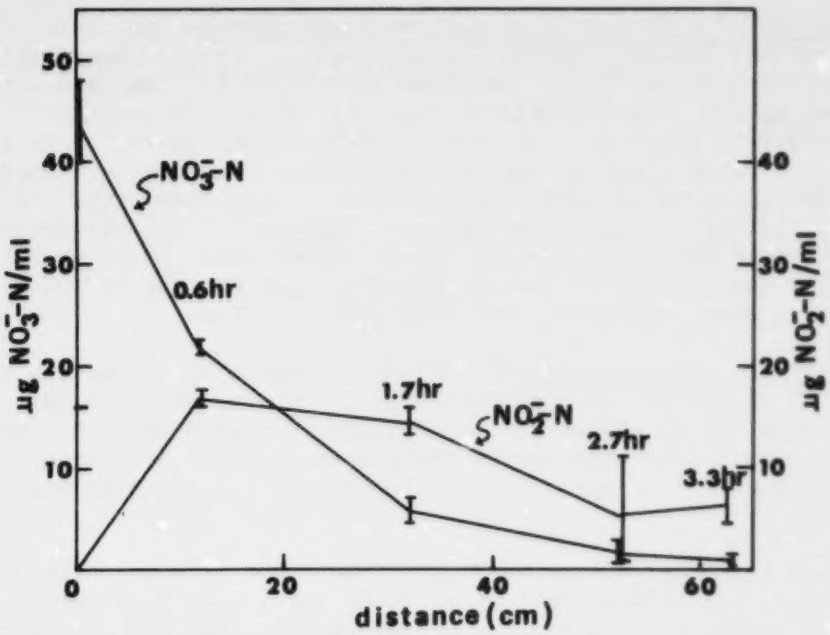


Figure A-59. Nitrate and nitrite levels (mean and standard error) and residence times in continuous flow columns at steady-state conditions (23°C).

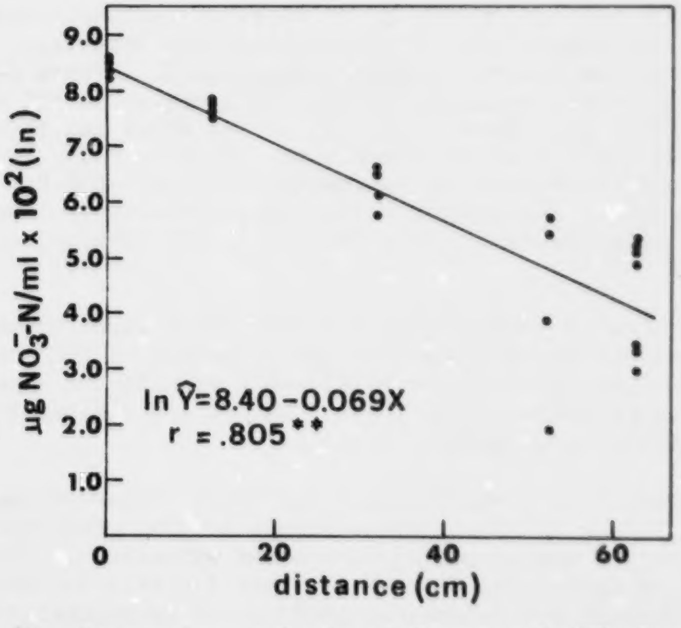


Figure A-60. Regression analysis of decrease in NO_3-N versus residence time (data in Figure A-59) in continuous flow columns (steady-state, 23°C).

TABLE A-127. OXIDATION-REDUCTION POTENTIALS (Eh_7) AT THREE LEVELS WITH CONTINUOUS FLOW COLUMNS (STEADY-STATE CONDITIONS, 23°C)

Level	Mean*	Range*
12 cm	+278	+250-+300
32 cm	+241	+227-+250
52 cm	+175	+160-+195

* millivolts

The data for gases detected during steady-state conditions (Table A-128) indicate that N_2, the end product of denitrification, was present in highest concentration. Small amounts of O_2 were also detected which probably resulted from trace contamination by air entering the columns when the influent bottles were changed. The presence of CO_2 and CH_4 indicated some heterotrophic activity was taking place within the columns, i.e., the organisms endogenous to the septic tank effluent were utilizing the small amount of organic C present in the influent as well as an carbonaceous by-products formed during the T. denitrificans metabolism. The neutralization reaction between the resulting sulfuric acid and limestone would also yield CO_2 (Burns, 1967).

Nitrous oxide, a recognized intermediate involved in T. denitrificans nitrate reduction, was detected only occasionally and not at all during steady-state conditions. The reason may be that in a semiclosed system such as in this study, any N_2O formed may be rapidly transformed to the next intermediate or to N_2. Nitric oxide, also a suspected intermediate, was never detected.

Hydrogen sulfide was detected after approximately four months continuous flow operation in concentrations ranging from 0.2 to 0.5 µmoles/cc gas produced. This was about the detection limit of the gas chromatrograph method used. Therefore, early production of trace amounts of H_2S would have gone undetected. Hydrogen sulfide production may have occurred in highly anaerobic micro-environments within the porous limestone permitting SO_4 to be reduced. Methane is also suspected to be produced in these microenvironments.

Sulfate was the primary end product detected in effluent from the columns (Table A-129). Sulfide was detected at low levels only at the bottom of the columns. These findings correlate with the redox potentials observed which were considerably higher than the -150 mv normally associated with SO_4 reduction (Patrick and Mahaptra, 1968). Only low levels of polythionates were present, most of which appeared to be thiosulfate. Other polythionates and possibly sulfide were present but could not be quantitated because of their low levels. Under optimal growth conditions, no sulfur intermediates are known to accumulate in T. denitrificans nitrate reduction but many external factors such as pH, O_2 concentration and phosphate concentration may affect the accumulation of intermediate (Vishniac, 1974). In these studies, TABLE A-128. CONCENTRATION OF GASES DETECTED IN CONTINUOUS FLOW COLUMNS (STEADY-STATE CONDITIONS, 23°C)

Gas	Mean*	Range*
O_2	2.40	1.20-3.60
N_2	43.10	41.30-44.70
CO_2	0.30	0.20-0.40
CH_4	t	t
N_2O	--	--

* μ moles/cc gas produced

TABLE A-129. MEAN CONCENTRATIONS (MG/L) OF SULFUR AND PRODUCTS IN CONTINUOUS FLOW COLUMNS (STEADY-STATE CONDITIONS, 23° C)

Level	$SO_4^{=}$-S	$S^{=}$	polythionates
Top	6.2	0.0	<0.5
12 cm	36.4	0.0	<0.5
32 cm	69.5	0.0	<0.5
52 cm	93.9	0.0	<0.5
Bottom	93.9	0.1	<0.5

conditions were such that substantial accumulation of the intermediates of S oxidation does not occur.

A regresssion equation relating the production of $SO_4^{=}$ to the decrease in $(NO_3^- + NO_2^-)$-N is shown in Figure A-61. The slope of the line (2.13) approaches the theoretical value of 1.90 for $SO_4^{=}$-S produced per NO_3^--N reduced. The difference in ratios may be due to the $MgSO_4$ formation. Precipitates should be negligible in this system since $CaSO_4$ has a solubility of 1.9 g/l at 20°C. The other sulfates probably have higher solubilities.

Table A-130 shows that a significant decrease in inorganic C occurred down the columns while changes in organic C content were insignificant. These data, in conjunction with the NO_3^- decrease and gas analyses data, indicate that the autotroph, _T_. _denitrificans_, is the dominant species within the columns.

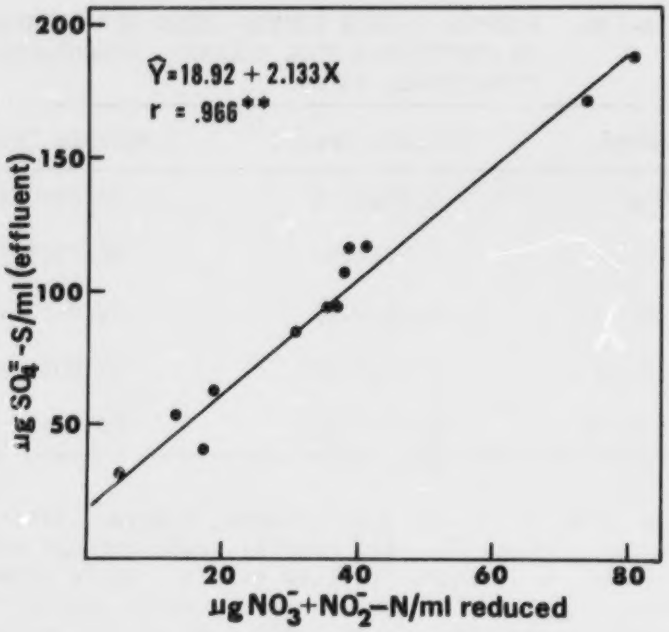


Figure A-61. Regression analysis of reduction of NO_3^- + NO_2^--N versus $SO_4^=$-S production in continuous flow columns (steady-state, 23° C).

The T. denitrificans nitrate removal system complies with most of the requirements for a denitrification system to function in conjunction with a septic tank seepage bed system. The rates are fast with complete dentrification occurring in less than 4 hr at 23°C. Assuming that the denitrification rate of 5°C is approximately 10% that at 23°C, complete denitrification at extreme winter temperatures could be obtained in approximately 1.5 to 2 days residence time.

The T. denitrificans system is also relatively inexpensive. An estimate for the amount of sulfur needed for a denitrification system serving a family of 4 to 10 years is approximately 450 kg of crude, lump S. The 1975 cost of this amount of S is approximately \$125 plus shipping. The cost of the limestone to mix with the S in sufficient quantities to obtain a reactor bed size to hold 2.0 days effluent (1500 L for a family of 4) would be \$90 plus shipping (approximately 4 m^3). Depending upon the type of septic system installed initially, this denitrification system would add approximately 20% to the total installation cost. Dividing the total cost by the number of years in use (estimated lifetime of 10 years) would be only \$20 per year.

The effect of the T. denitrificans nitrate removal system upon the performance of the overlying seepage bed can only be determined in field studies as well and the amount of maintenance necessary for proper functioning of the system. It appears that, after establishment of the T. denitrificans on the S, little maintenance would be needed.

TABLE A-130. SOLUBLE CARBON LEVELS (MEAN AND STANDARD ERROR) IN CONTINUOUS FLOW COLUMNS (STEADY-STATE CONDITIONS, 23°C)

Level	Soluble Org. C	Soluble Inorg. C
Top	6.25±0.75	65.25±0.25
12 cm	5.50±0.50	56.75±0.75
32 cm	5.75±0.75	51.25±0.75
52 cm	5.00±0.50	50.00±0.50
Bottom	6.50±0.00	50.50±0.50

The resulting level of SO_4 in the effluent, however, limits the application of this system. Since SO_4 will readily leach through most soils and reach the ground water, a potential problem exists. Quite often, homes using on-site waste disposal also have their own water supply from drilled wells on their property and the problems related to excessive levels of SO_4 in drinking water are well known. Whether or not a level of sulfate sufficient to render the drinking water undesirable is reached in ground water below a T. denitrificans nitrate removal system depends upon the density of systems per unit area and the volume and recharge characteristics of the aquifer or watershed. Walker, et al., (1973b) speculated that if the density of residences using home sewage disposal was less than one per 3 hectare, NO_3 itself would not pose a problem mainly because of dilution. A similar siutation would apply to SO_4.

Little discussion has been devoted to the possible production of $S^=$ from the resulting SO_4 in soils. This appears to be much less of a problem than SO_4 because of the extreme conditions necessary for the SO_4 reduction. Highly anaerobic conditions (Eh -150 mv) and the presence of organic matter are necessary for $S^=$ production and even if these conditions do occur, the chances of $S^=$ moving considerable distances in soils are low because of ferrous sulfide (FeS) precipitations (Patrick and Mahapatra, 1968). Data from this study show that $S^=$ does not appear in the S-limestone columns as long as residence times are near that necessary only for complete denitrification. Nitrate acts as an oxidant and inhibits the formation of $S^=$ (Engler and Patricks, 1973) probably by posing the redox potential above -150 mv.

The best procedure to establish the organims T. denitrificans upon the sulfur within a NO_3 removal system still needs to be resolved. The organism is ubiquitous in soils and will become dominant when anaerobic conditions are maintained in the presence of S and NO_3 in soils (Mann et al., 1972). The organism is also found in canal waters, mine waters and marine sources (Vishniac, 1974). If the organism is not indigenous to the septic tank effluent, it would appear that the organism could easily be cultured commercially. A freeze-dried package could then be added to the sulfur denitrification bed and, under the proper conditions, the organism should flourish.

In summary, the sulfur T. denitrificans nitrate removal system would have only limited use in connection with septic tank seepage bed home waste disposal systems. The major limitation is the final SO_4 concentration.

Preliminary field denitrification system--A denitrification-P removal system was installed at the University of Wisconsin Experimental Farm at Arlington (site J). The design of the field system (Figure A-62) included a wet well used to collect and hold approximately 280 L (75 gallons) of nitrified effluent from either the septic tank-sand filter system or the aerobic unit-sand filter system. A small laboratory pump controlled by a timer switch was used to inject ≈ 100 ml of 30% methanol (twice the stoichiometric amount based on 40 mg/L of nitrate-N) into the 280 L (75 gal) of nitrified effluent. Twelve minutes after the methanol injection, a time delay switch activated a submersible pump which pumped the mixture of nitrified effluent and methanol into a sealed 1.2 m (4 ft) diameter dentrification tank. The denitrification tank was filled with 0.9 cm (3/8 in) "lannon" stone (a carbonate rock). The void volume of liquid capacity of the denitrification tank with the stone in place was 280 L (75 gal). The lannon stone was chosen to determine whether phosphorus could be removed along with nitrate-N.

The methanol and effluent mixture was distributed into the denitrification tank via a 5 cm (2 in) manifold located at the bottom of the tank. This forced the liquid already present in the tank out via another 5 cm (2 in) manifold located at the top of the tank. The effluent from the denitrification tank was then sampled. A check valve was installed in the submersible pump line to prevent backflow from the denitrification tank into the wet well.

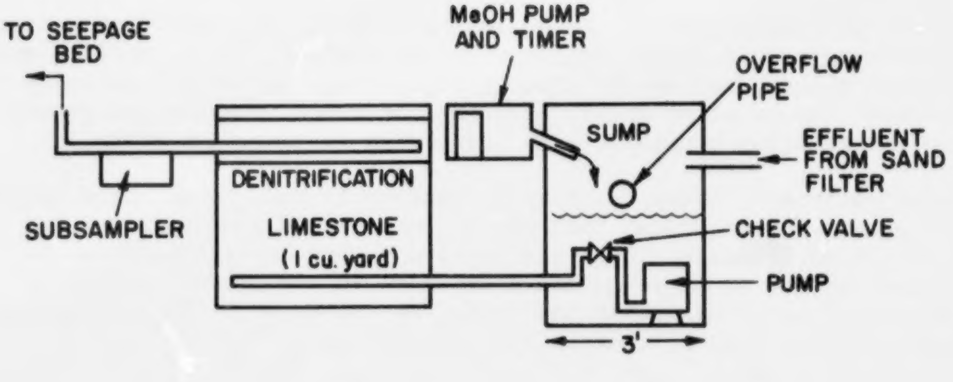


Figure A-62. Field denitrification system - site J.

The entire sequence of events was controlled by a predetermined timing device. Experiments were performed at 12 and 24 hour time cycles. For instance, if a 12 hour time cycle was employed, effluent was collected in the wet well for 12 hours. If a volume over 280 L (75 gal) flowed into the wet well, the excess flow was bypassed through an overflow. The 280 L (75 gal) in the wet well was then sent through the cycle described above. However, if less than 280 L (75 gal) flowed into the wet well, the cycle was not activated and no methanol-effluent mixture was pumped through the denitrification unit. For the 24 hour time cycle, the 280 L collected in the wet well was cycled through the system once per day.

Twenty-four hour composite samples were obtained from the septic tank, aerobic treatment unit, intermittent sand filters, ultraviolet light irradiation unit and denitrification unit.

An analysis of the effluent quality data for the denitrification unit is presented in Table A-131. Septic tank-sand filter effluent was directed to the unit from June, 1976 through August, 1976 while aerobic unit-sand filter effluent was treated by the denitrification unit through October, 1976. There was little difference in these effluents with respect to P and N. The value of the nitrate-N concentration of the sand filters effluents ranged from 29 mg/L to 55 mg/L over the experimental period. The log mean nitrate-N concentration of the denitrification unit effluent was 3.7 mg/L. This shows that a substantial reduction in nitrate-N was attained, although a sample from one day reached a value of 30.7 mg/L.

The experiments were set on a 12 hour cycle; however, due to low flow conditions 280 L (75 gal) per period 24 hour and 36 hour cycles occurred. The amount of methanol addition (≈ 100 mL per 75 gallons) resulted in significant denitrification rates. Concentrations of BOD_5 in the denitrification unit effluent ranged from 1 mg/L to 4 mg/L. This indicated that excess carbon concentrations due to the methanol overdosing were not occurring in the unit.

Phosphorus concentrations in the denitrification unit effluent were not significantly reduced from the influent concentrations. This confirms earlier findings that phosphorus removal after a significant period of operation was not successful due to slime growth covering the sorption sites and organic anions in the effluent competing with phosphorus for sorption sites.

Concentrations of total coliforms, fecal coliforms, fecal strep and TSS also were monitored. Although both septic tank-sand filter effluent and aerobic unit-sand filter effluent was applied to the denitrification unit, it appears that neither an increase nor a decrease in the concentrations of bacteria and TSS occurred within the unit. An increase in TSS within the unit and clogging of the denitrification bed was originally feared.

Although the operation of the denitrification unit was rather complicated, no mechanical breakdown of the equipment occurred. Only minor adjustments were made to the timer control and the methanol injection system to insure proper feeding rates. Clogging of the submerged denitrification bed or periodic suspended solids upsets were not experienced over the experimental period.

TABLE A-131. PRELIMINARY RESULTS FOR FIELD
DENITRIFICATION UNIT - SITE J
(JUNE - OCTOBER, 1976)

Parameter	Data	Sand Filter	Denitrification Unit
Ammonia-N	Mean	.1+	0.1
	Range	0.1 - 0.5	0.1
	# Samples	12	
Nitrate + Nitrite-N	Mean	34	3.7*
	Range	29 - 55	0.2 - 30.6
	# Samples	12	7
Total Phosphorus-P	Mean	15	14
	Range	12 - 19	11 - 16
	# Samples	10	6
Orthophosphate-P	Mean	13.5	12
	Range	11 - 16	8 - 15
	# Samples	12	7

* Log normal distribution

+ All data in mg/L

Ion Exchange/Ammonia Removal--

The ion exchange procedures envisioned for nitrogen removal for small flows employed a column operation in which septic tank effluent is passed under pressure through a column of ion exchange resin. After breakthrough of nitrogen occurred, it was envisioned that the resin column would be replaced with a regenerated module. Regeneration would take place at a centralized location where it would be economical to recycle the resin. To be practical, columns would need to be regenerated no more than once per month by a firm contracted to collect and regenerate the columns for the individual on-site systems.

There is sufficient experimental evidence to indicate that clinoptilolite will effectively remove ammonia-nitrogen from secondary effluents. There has been little experience, however, with the application of septic tank (or primary) effluents to clinoptilolite resins. This preliminary phase of the research effort was aimed at evaluating the exchange capacity of several particulate sizes of clinoptilolite treating septic tank effluents. Operation and maintenance problems were also to be evaluated in this work.

Continuous flow columns--Laboratory experiments were conducted using 3.8 cm glass columns outfitted with a fritted glass support on the bottom. Specially prepared clinoptiololite (20 x 40 mesh) was placed in the column. The 120 grams of clinoptilolite occupied approximately 180 cm^3 to a height of 17 cm above the base. Feed to the columns was provided through Marius bottles located above the ion exchange columns, and flow was by gravity through the resin. Flow was regulated at 10 bed volumes per hour. Effluent was collected in carboys below the columns.

In the initial experiments, two columns were used; one was fed distilled water supplemented with chloride salts (synthetic waste). The remaining column received septic tank effluent from site J.

The clinoptilolite was prepared for ion exchange by equilibrating the raw resin in 1 M NaCl for 2 days. The clinoptilolite was then washed with distilled water until no traces of chloride were detected in the elutriate. The washed resin was air dried overnight and then oven dried at 105°C for 24 hours. The resin was then hand sieved and appropriate fractions withdrawn for use in the columns.

The influent to and effluent from the ion exchange columns were analyzed routinely for Na^+, NH_4^+, Ca^{++}, Mg^{++}, K^+ and pH. Batches of septic tank effluent were collected at site J and stored at 4°C for not more than 7 days. Effluents from the columns were removed daily and aliquotes stored at 4°C. Samples for NH_4^+ analyses were treated with H_2SO_4 to a pH of approximately 3.0.

Preliminary experimental results--The influent cation composition to the exchange columns for a septic tank effluent and a "synthetic" waste is shown in Table A-132. Selected breakthrough curves for septic tank effluent and the "synthetic" waste are shown in Figures A-63 and A-64. The clinoptilolite was originally placed in the sodium form so that Na^+ is exchanged for the other cations. The breakthrough curves conform to the selectivity of cations by clinoptilolite:

$$K^+ \geq NH_4^+ > Ca^{++} > Mg^{++}$$

During the tests, the exchange column receiving septic tank effluent became progressively blackened as organic matter moved through the column. Substantial head loss occurred during this period and adjustments were made to insure constant flow rates. Severe clogging did occur in the column shortly after breakthrough occurred.

Results of the ammonium exchange capacities for the septic tank effluent and the "synthetic" waste are presented in Table A-133.

TABLE A-132. INFLUENT QUALITY TO ION EXCHANGE COLUMNS

Parameter	Septic Tank	Synthetic Waste
NH_4-N(mg/L)	33	31
Ca^{++}(mg/L)	54	30
Mg^{++}(mg/L)	30	32
K^+(mg/L)	13.4	12.5
pH	7.4-7.6	6

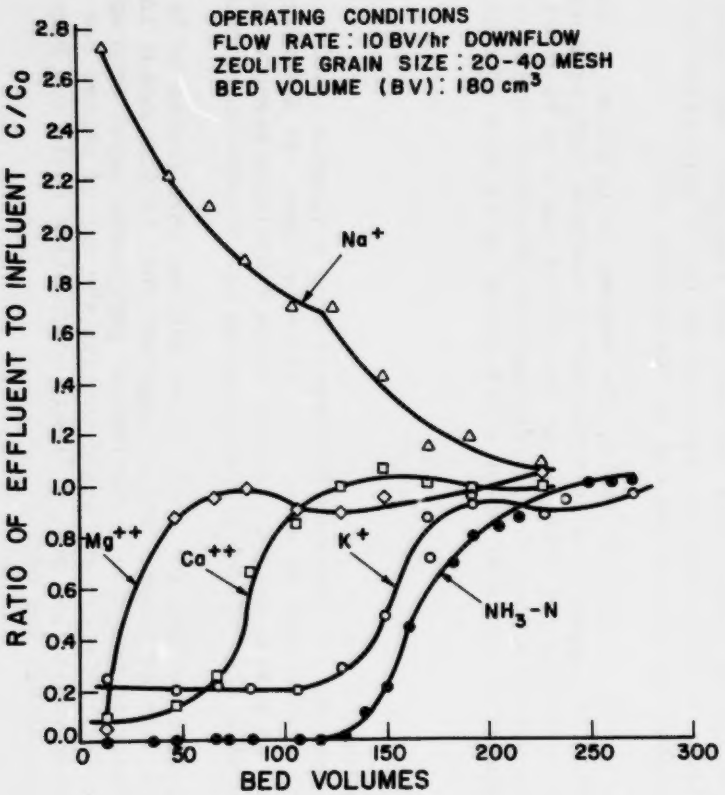


Figure A-63. Breakthrough curves for clinoptilolite - laboratory study

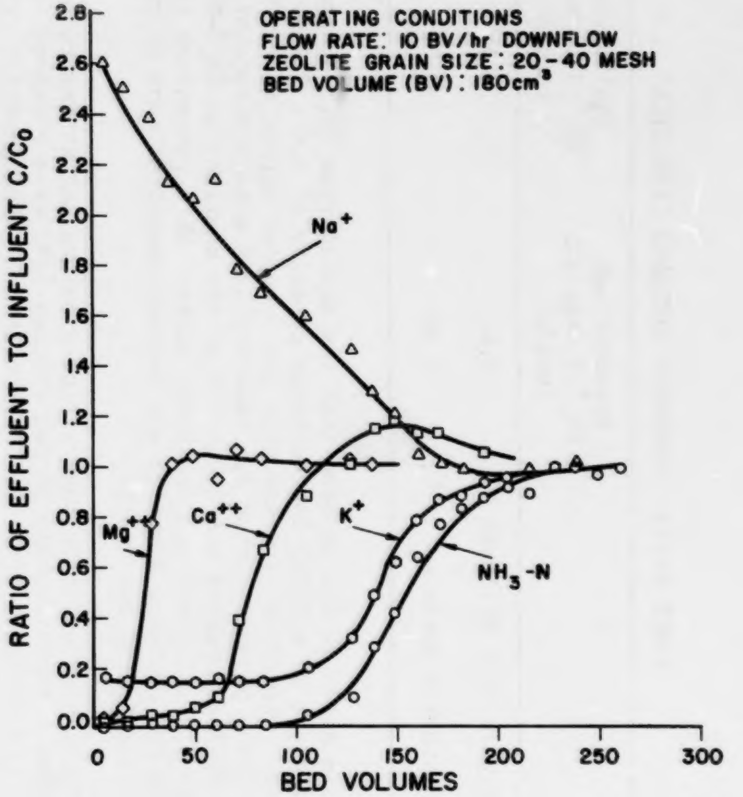


Figure A-64. Breakthrough curves for clinoptilolite - laboratory study.

TABLE A-133. AMMONIUM EXCHANGE CAPACITIES

	Breakthrough NH_4^+-N Capacity (meq/g)	Equilibrium NH_4^+-N Capacity (meq/g)
Septic Tank Effluent	0.41	0.54
Synthetic Waste	0.39	0.52

From Table A-133 it can be seen that the equilibrium NH_4^+-N capacity is about 0.5 meq/g out of a total ion exchange capacity of 1.6 - 2.0 meq/g for clinoptilolite. Therefore, the ammonium ion occupies approximately 30% of the exchange sites at equilibrium for this influent cation composition. The breakthrough NH_4^+-N capacity was about 75% of the equilibrium NH_4^+-N capacity for the septic tank effluent and synthetic waste. This percentage is dependent on the ion exchange kinetics and process variables such as clinoptilolite size, flow rate and bed depth.

Regeneration of the septic tank effluent column was attempted using a 5% NaCl solution. Approximately 30 bed volumes (BV) were required to completely elute the ammonium from the clinoptilolite. Regeneration effectively removed most of the organic matter and suspended solids in the bed. Another exhaustion cycle using septic tank effluent was then performed using the regenerated bed. Breakthrough and equilibrium NH_4^+-N capacities were essentially the same for the second run as the initial run.

Based on these preliminary tests, the removal of nitrogen as ammonia using clinoptilolite appears to be technically feasible, but exchange capacities are less than desirable requiring large amounts of resin to be practical. In-house waste modifications through segregation practices coupled with ion exchange treatment of the grey water would appear to be more appealing at this time.

Phosphorus

Chemical Processes--

Current methods for removing phosphorus from effluents involve the addition of coagulating agents such as alum, iron salts or lime. As discussed earlier, waste may also be passed over a granular media containing these coagulants which in turn will also effect a precipitation and absorption of phosphorus.

Chemical feed devices for small flows systems have proved to be maintenance headaches. In addition, pacing coagulant feed to phosphorus input is complicated owing to the wide variations of phosphorus input from an individual home. A more desirable concept would involve the use of packed bed systems where wastewater would flow past a fixed, coagulating media and precipitate and be absorbed within the bed.

Limestone Column Laboratory Studies--A gravity flow laboratory system was employed to evaluate the effectiveness of dolomite or calcite in removing phosphorus from septic tank effluent. Details of the experimental apparatus appear in the preceding section on nitrogen removal systems. Table A-134 shows the P removal results for all systems studied. Figure A-65 is data taken from the 0.64 cm (0.25 in) calcite column and is indicative of results obtained in the other studies. Therefore, graphs of the other column systems are not presented.

The P removal results obtained for a 4 month study using 0.64 cm (0.25 in) dolomite (Table A-134) indicated effective P removal during the first 2 months. After this time, however, the effectiveness of the system declined considerably. The P removal for the series of columns was 70% for the first month, but decreased to approximately 8% in the third and fourth month. The total P removal for the 4 months average 25%. The results from the columns containing 0.96 cm (0.38 in) dolomite chips indicate that the P removal for the first month was quite good. However, after this time the system rapidly lost its ability to sorb P. The percent of P removal was 54% for the first month of operation, but decreased to 8% by the fourth month. The total P removal for the 4 months of operation averaged 23%, a slight decrease from that removed in the 0.64 cm (0.25 in) chips for the same length of time.

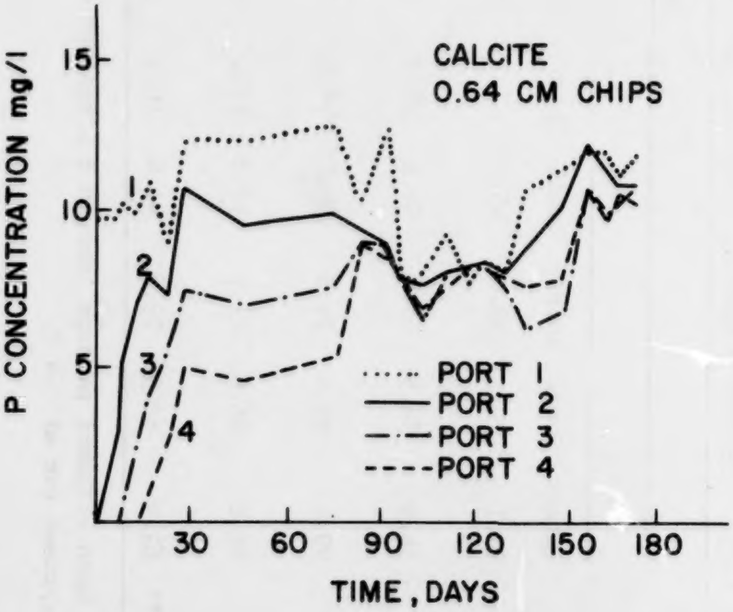


Figure A-65. Phosphorus concentration as orthophosphate in samples taken from ports in 0.64 cm (0.25 in) calcite column series. Port 1 follows the sand column, port 2 follows the first limestone column, port 3 follows the second limestone column (methanol added), and port 4 follows the third limestone column.

TABLE A-134. PERCENT P REMOVAL (INTERVAL AND CUMULATIVE) FOR CALCITIC AND DOLOMITIC LIMESTONES*

Limestone System	Months											
	1		2		3		4		5		6	
	Interv	Cumul	Interv	Cumul	Interv	Cumul	Interv	Cumul	Interv	Cumul	Interv	Cumul
Dolomite 0.64 cm	70.1	70.1	20.0	45.7	7.3	20.3	9.7	24.7	--	--	--	--
Dolomite 0.96 cm	54.5	54.5	14.6	34.1	18.8	28.9	7.6	23.0				
Calcite 0.32 cm	99.0	99.0	74.5	86.5	46.6	72.9	26.9	63.7	25.9	56.0	11.5	52.2
Calcite 0.64 cm	86.6	86.6	61.5	72.8	47.1	63.9	14.2	53.6	18.4	47.2	4.8	41.7
Calcite (6°C)**	62.9	62.9	23.8	41.2	16.5	32.6	10.7	27.3	--	--	--	--

* Data based upon effluent passage through 3 limestone columns equalling a 3 day residence time.

** All other systems run at 20° C.

The 0.32 cm (0.13 in) calcite column system proved to be the most effective for removing P of all the systems tested (Table A-134). However, this system also lost considerable P removing ability near the end of the 6 month experiment. This series removed 99% of the P entering the columns in the first month, but P removal decreased to 12% in the sixth month of operation. Phosphorus removal was 64% for the first 4 months of the study and 52% for the entire 6 month period. The column series with the 0.64 cm (0.25 in) calcite chips was less effective than the columns using 0.32 cm (0.13 in) calcite chips but more effective than either of the dolomite series. Phosphorus removal was 87% for the first month, but decreased to 5% for the sixth month. The percent removal for the first 4 months was 54% while for the entire 6 month operation 42% was removed.

Using 0.64 cm (0.25 in) calcite chips, a decrease in temperature from 20 to 6°C resulted in less P sorbing capacity in both the sand and the calcite. The percent removal was 63% for the first month, 24% for the second month, and only 11% for the fourth month. The total percent removal for the 4 month study was 27% as compared to 54% for a similar system at 20°C.

Several conclusions can be gleaned from these results. First, increasing the residence time from 1 to 3 days removes more P from septic tank effluent (Figure A-65). Also, calcitic limestone is superior to dolomitic limestone in P removing ability. However, the ability of either limestones to remove P from septic tank effluent for several months is limited. Over several months of operation the P sorption capacity decreases markedly. This decrease in P removal appears to be due to organic anions in the effluent competing with P for sorption sites plus microbial growth physically blocking sites.

Organic anions such as glutamate and acetate compete with P for sorption sites, decreasing the P removal ability of the limestones (Struther and Sieling, 1950). In a separate study calcite columns were dosed daily with an NaCl solution containing P equaling the ionic strength and P concentration of septic tank effluent. The average cumulative P removal of 80% for a 6 month study was obtained (Bent, 1975). At the termination of the U.W. experiment, the columns were disassembled and considerable slime growth was noted. This was especially true for the second column in each series where the majority of the denitrification occurred. However, the data did not indicate excessive inhibition of P removal in the second column versus the others. Therefore, it appears the slime growth and/or organics hamper the P removing ability of the limestones. Unless some means could be developed to inhibit slime growth or remove organics in the effluent, the success of P removal using limestones would only be temporary.

Another conclusion is that the smaller size limestone is more effective for P removal, undoubtedly due to the fact that smaller chips offer a greater total surface area. Smith, et al., (1971), however, found that clogging increased with smaller sized bed materials. In this light, a conflict develops. If a system is to function for both N and P removal, the surface area of limestones must be sufficient for effective P removal and denitrification without excessive clogging. Although methods are available to remove the clogged portions and gas build-up, the frequency of treatment would be less if larger-sized chips were employed.

Field denitrification phosphorus removal study--In an effort to demonstrate the effectiveness of a limestone filter bed system for phosphorus and nitrogen removal a field unit was installed at site J (Figure A-62). Details of this system were presented in the nitrogen section. As indicated, phosphorus removals were poor due to the development of biological growths on the limestone and possibly due to competitive reactions at the sorption sites by organic anions.

DISINFECTION OF WASTEWATER

The investigations on sand filters and other on-site treatment systems have demonstrated that effluents of relatively high quality can be produced on site. There are, however, still questions as to the safety of discharging this treated effluent to surface waters or on the land. Some discharges may require the use of disinfection processes to further reduce the risks of infectious disease. The following review of disinfection looks at some of the most promosing disinfectants for on-site use and discusses some of the factors which affect their usefulness.

Factors Affecting Disinfection

Disinfection is a physical-chemical process conducted on biological systems and as such is governed by the laws of chemistry and physics but is complicated by the large number of biological systems and possible biochemical reactions. Early in the study of disinfection it was shown that the destruction of cells by chemical agents occurs at some finite rate (Chick, 1908). What controls this rate has been the subject of a considerable amount of research. Also important is the level of disinfection because, generally, at some point in time the rate becomes zero. Some of the most important factors are: 1) nature and chemistry (or physics) of the disinfectant, 2) concentration (or intensity) of the disinfectant, 3) the nature of the microorganisms being disinfected, 4) number of microorganisms, 5) presence of interferences in the wastewater, 6) time of contact between disinfectant and microorganism and 7) the temperature of the system (Fair, et al., 1968). This list alone shows the complexity of the process; however, the first simplifying assumption is that these will be discussed in terms of their relationship to small wastewater flows.

Nature of the Disinfectant--

A list of potential disinfectants (Table A-135) shows a considerable variation in the types that could be used for small waste flows. The ideal disinfectant should meet the following criteria: 1) effective against infective organisms at reasonable concentrations, 2) relatively unreactive with other constituent in wastewater, 3) safe to handle in concentrated forms, 4) easy to administer reliably, 5) economical, and 6) not a pollutant in itself (ASCE, 1970). Some of the disinfectants in Table A-135 can be immediately ruled out by these criteria. Quaternary ammonium (cost), dyes (effectiveness), detergents (effectiveness), hydrogen peroxide (effectiveness), X-Rays (safety and cost), ultrafiltration (hard to administer), freezing (effective, cost, hard to administer) and ultrasonics (cost, effectiveness) are not presently practical alternatives for disinfection of small wastewater flows.

TABLE A-135. POTENTIAL DISINFECTANTS FOR SMALL WASTE FLOWS

Chemical	Possible Mode of Action
1. iodine	oxidation
2. chlorine & compounds	oxidation, enzyme inactivation
3. bromine & compounds	oxidation
4. ozone	oxidation
5. heavy metals	enzyme inactivation
6. dyes	protein denaturation
7. soaps & synthetic detergents	destruction of cell membrane
8. quaternary ammonium compounds	destruction of cell membrane
9. hydrogen peroxide	oxidation
10. alkalies and acids	protein denaturation
11. formaldehyde	protein denaturation
Physical and Mechanical	
1. heat	protein denaturation
2. ultra filtration	removal from liquid medium
3. freezing	osmotic pressure
Radiation	
1. U.V. light	DNA damage
2. X-rays	molecular action
3. Ultrasonics	cell disruption

Where the other disinfectants rate in relation to the above criteria has been the subject of considerable debate. As of yet, no disinfectant is known to adequately meet all criteria under all conditions. The alternative then is to outline the proper conditions for application of a particular disinfectant.

Probably the most important disinfectants for small flows are: chlorine, ozone, ultraviolet light and iodine, in that, more is known about these methods and equipment exists for their application to small flows. Chlorine is the most popular disinfectant because it has been demonstrated to be quite effective, and it is relatively cheap. However, it is sensitive to pH changes in the normal range of 6-8 and has been shown to be toxic to aquatic life in low concentrations (when combined with wastewater). Iodine undergoes many of the same reactions as chlorine but at different rates and at different pH values and iodine is unreactive with ammonia. Some studies have indicated that iodine is superior to chlorine as a wastewater disinfectant but the cost is considerably greater. Little is known about the toxic effects of iodine and related compounds. Use of ozone has created new interest, and many studies have alluded to its superiority over chlorine. However, ozone is more sensitive to oxidant demanding materials in wastewater and presently its application is much more expensive than chlorine (Budde, et al., 1977). Ultraviolet light has also been shown to be an extremely effective disinfectant, giving not only a rapid rate of kill but also providing a high level of disinfection (Huff, 1965). However, the required equipment is expensive and its effectiveness is inhibited by turbidity and other potential interfering wastewater constituents.

Concentration of the Disinfectant--

As the concentration (or intensity) of a disinfectant increases both the rate and level of microbial kill increase. The goal of disinfection is to obtain a desired level of kill in a reasonable length of time with the least amount of disinfectant. Because of the many interferences in wastewater, it is very difficult to control the level of active disinfectant. The term "concentration of disinfectant" should not be confused with "dosage of disinfectant," as a great percentage of the dosed disinfectant may be immediately converted to ineffective forms. A disinfectant "residual" should be maintained throughout the contact period.

Nature of the Microorganisms--

There is an extreme variation in the rates at which various groups of microbes are inactivated by a given disinfectant at constant conditions or in the required concentration of active disinfectant to achieve a given level of reduction in a given time.

The organisms of primary interest in wastewater disinfection are bacteria, virus and amoebic cysts. Bacterial spores are generally the hardest to inactivate while young, growing bacteria are the easiest. Somewhere in between are viruses and cysts. Of particular concern is the use of bacterial indicators to evaluate the efficiency of disinfection, as an effluent made devoid of coliforms may still harbor large numbers of viruses or cysts. There is, presently, no good indicator of adequate disinfection.

Number of Microorganisms--

Because disinfection is a rate process, the larger the number of organisms, the longer the time to reach a desired level in the effluent. Appendix C reports on the numbers of selected bacteria normally found in typical small wastewater flows.

Interferences in the Wastewater--

Wastewater is known to contain many materials which may interfere with the process of disinfection. Materials which react with the disinfectant before it has a chance to act on microorganisms obviously affect the disinfection process.

A primary example of interference is the reaction of chlorine with ammonia to form chloramines. Although chloramines have been shown to have disinfecting properties, they react at a much slower rate than chlorine. The presence of suspended solids interfere with the penetration of ultraviolet light and may also shield the microorganisms against chemical attack. An increase in organic material more severely affects the demand for ozone than that for chlorine. The pH of a wastewater has a dramatic effect on the action of free chlorine which is much more effective at pH less than 7 than at pH greater than 8.

Comparisons of the average characteristics of various small wastewater flows show that sand filtered effluent would be the easiest to disinfect because it contains the lowest level of interferences. It should also be noted that there can be large variations in effluent quality for a particular type of effluent. This increases the difficulty of controlling disinfection.

Contact Time--

Since disinfection is a rate process, the longer the contact time between disinfectant and microorganisms, the greater the percentage of kill. It does not necessarily follow that holding a mixture of disinfectant and wastewater for long periods of time will lead to high levels of disinfection. Many of the factors listed above may act to decrease the rate of disinfection to zero after a relatively short time. In general, the amount of contact time required to achieve wastewater disinfection using ultraviolet light is in the order of magnitude of seconds. Ozone requires longer periods although less than chlorine and iodine.

For small flows, it is fairly easy to construct contact tanks giving extended contact times up to 24 hours. The primary cost of such installations is that for labor.

Temperature of the Wastewater--

Chemical disinfection proceeds at a slower rate at lower temperatures. Contact times should be increased to account for cold weather flows. It is not clear how cold temperatures affect ultraviolet light disinfection.

Summary--

All the above factors can be controlled to some extent thus improving the chances for successful disinfection. It is important to know how a particular disinfectant reacts in the particular wastewater being treated, what the potential variability of the wastewater is and what potential health hazards exist for each particular waste. One conclusion which can be reached from the above discussion is that sand filter effluent should be the easiest to disinfect not only because many of the interferences have been removed, but also because the number of microorganisms is less. The following discussion will consider several disinfectants and how they might react in sand filtered effluent.

Disinfectants for Small Flows

As can be seen in Table A-135 there are many possible disinfectants which could be applied to small flows; however, only a few are presently practical for use. Although this section discusses only the most promising, it is not implied that these are the only choices. The disinfectants chosen for discussion were done so on the basis of the disinfectant presently being available at reasonable costs and the availability of reliable feeding equipment.

Chlorine--

Chlorine remains a disinfectant of choice because it is readily available and it is the least expensive. More is known about chlorine as a disinfectant than any other. The primary disadvantages of its use are its reactivity with wastewater interferences, and the toxic effects of its residual to other aquatic life forms.

Chlorine can be fed as a gas (Cl_2), a liquid NaOCl and Ca (OCl_2) or a solid (Ca $(OCl)_2$). Because of the potential safety hazard of feeding chlorine gas, the use of hypochlorites is a better choice for small flow use. Hypochlorites are easier to transport and to store and use in remote locations. However, they are more expensive than elemental chlorine. The most common type of chlorine feeder for small wastewater flows is the solid feeder. Flow past chlorine tablets dissolves them at a slow rate. Dosage can be controlled either by flow-control weirs or by diverting only a controllable portion of the flow through the unit.

A portion of the chlorine, when added, is rapidly utilized as a "chlorine demand." These are the side reactions with interferences. It is generally felt that disinfection by chlorination is most effective when a residual of free chlorine is present after the designed contact time. In larger scale installations continuous monitors have been used to closely control the residual levels after disinfection to limit the amount being discharged into receiving waters. These, however, would be much too expensive for small scale use. The only alternative to assessing the effectiveness of a chlorination system is to take occasional grab samples and measure the free chlorine level. The dilemma is to have a residual to demonstrate the efficacy of the process but not to have a high level of residual which might prove toxic to other forms of aquatic life.

A dry feed chlorinator was tested at two of the field sites (E and H) described previously. Sand filter effluent flowed through the chlorinator and then into a 0.57 m^3 (150 gal) contact chamber at site E and a 0.43 m^3 (115 gal) chamber at site H. These volumes provided detention times of from 3 to 21 hours. The flow was directed past one stack of 7 cm (3 in) diameter calcium hypochlorite tablets. Dosage was found to be a function of flow as is shown in Figure A-66. It ranged from 7 mg/L to 40 mg/L at site E and average 18 mg/L at site H.

The effect of sand filtration and chlorination on the reduction of indicator bacteria in the treatment unit effluents is shown in Table A-136 and A-137. As would be expected, the higher dosage (lower flow) at site E resulted in slightly higher kills, yet, with the exception of total bacteria, these kills were not significantly higher at the 95% confidence level. Of the indicators, fecal and total coliforms appeared to be more sensitive to destruction by chlorination (the highest percentage reduction), while total bacterial counts were least affected. A substantial portion of the reduction of bacteria can be attributed to removal of sand filtration. No tests were run to determine the impact of these chlorinators on the level of viruses in effluents.

The major maintenance problem experiences during the use of these chlorinators was the failure of the tablets to move downward as the bottom tablets dissolved. This was caused by a disintegration of the upper tablets due to high humidity in the chlorinator. At other times, this disintegration caused parts of the tablet to slake off and thus produce excessive dosages of chlorine (the highest measured residual was over 20 mg/L).

The chlorine tablets utilized in the chlorinator cost about $4.20 per kilogram ($1.98/lb). If one considers an average dose of about 20 mg/L, then

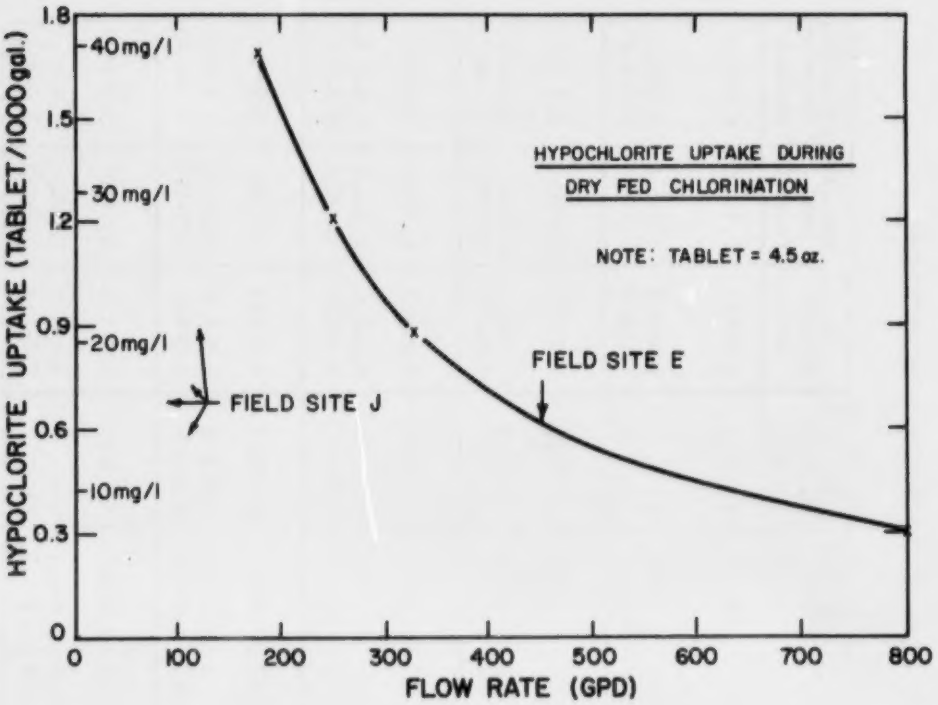


Figure A-66. Hypochlorite uptake during dry feed chlorination.

it would cost about $23 per year to treat an average flow of 0.76 cubic meters per day (200 gpd).

Iodine--

It has been suggested that iodine is a better wastewater disinfectant than chlorine because it is less reactive to interferences, nonspecific reactions are slower, and it remains in the elemental state at normal pH values and it is unreactive with ammonia (ASCE, 1970). Like hypochlorite, it is relatively easy to transport, store and use. Some of the disadvantages are that little is known about the required doses for sand filter effluent or about its toxic effects.

Iodine is normally fed as a solution (See Figure A-67 for a configuration of a typical iodine saturator), but since it is sparingly soluble in water, large amounts of solution may be required to achieve proper dosage in the effluent. The solubility is affected by both pH and temperature.

Although the cost of iodine ($6.00/kg) makes it too expensive for large scale treatment, the advantages of transport and storage may outweigh chemical

TABLE A-136. BACTERIAL LEVELS FOLLOWING SAND FILTRATION AND CHLORINATION

Site/Date System	E Oct 1973-May 1974			E Aug 1974 - Mar 1975			H Sep 1973-Feb 1975		
Parameter	Septic Tank	Sand Filters	Chlorine Contact	Septic Tank	Sand Filters	Chlorine Contact	Aerobic Unit	Sand Filters	Chlorine Contact
Total Bacteria, log #/L	9.8(14)	8.7(10)	3.5(12)	9.3(9)	7.8(9)	6.0(11)	8.5(45)	7.8(40)	6.6(42)
Mean (# of Samples)									
Coeff. of Variation									
95% Conf. Int.	9.6-9.9	8.2-9.1	8.0-8.8	8.9-9.5	6.9-8.8	5.0-6.9	8.4-8.6	7.5-8.1	6.1-7.0
Range	6.7-10.1	7.8-9.5	6.0-9.0	8.5-9.6	5.7-9.5	3.0-7.8	7.5-9.1	6.5-9.2	4.6-9.2
Total Coliforms, log #/L									
Mean (# of Samples)	7.3(15)	5.2(11)	3.3(13)	7.0(12)	4.1(12)	1.5(11)	6.2(50)	5.2(44)	2.5(51)
Coeff. of Variation									
95% Conf. Int.	7.1-7.5	4.3-6.1	2.0-4.6	6.7-7.2	3.3-5.0	0.7-212	6.0-6.4	4.9-5.3	2.0-3.1
Range	6.6-7.9	3.3-6.8	0.7-5.9	6.3-7.4	3.0-6.9	0.5-3.7	4.8-6.9	3.8-7.1	0.3-5.7
Fecal Coliforms, log #/L									
Mean (# of Samples)	6.7(14)	4.7(11)	2.8(15)	6.8(14)	3.8(13)	1.3(11)	5.3(51)	4.3(49)	1.9(51)
Coeff. of Variation									
95 % Conf. Int.	6.4-7.0	3.7-5.7	1.7-3.9	6.6-7.0	3.0-4.7	0.7-2.1	5.1-5.4	4.0-4.6	1.5-2.3
Range	5.0-7.4	3.0-6.6	0.3-5.4	6.1-7.3	1.0-5.6	0.7-2.1	4.3-6.4	3.0-5.6	0.3-4.1
Fecal Strep, log #/L									
Mean (# of Samples)	4.5(15)	3.3(11)	2.1(14)	4.5(14)	2.8(14)	1.3(12)	4.4(52)	3.7(50)	1.9(52)
Coeff. of Variation									
95% Conf. Int.	4.2-4.8	2.3-4.0	1.3-3.0	4.1-4.8	2.0-3.2	0.8-1.8	4.2-4.6	3.2-4.1	1.5-2.2
Range	3.3-5.3	1.3-4.2	0.3-4.5	3.0-5.3	1.3-5.7	0.3-3.2	2.3-6.1	1.5-5.3	0.3-4.8
Ps. aeruginosa, log #/L									
Mean (# of Samples)				4.5(13)	2.4(12)	<1.3(9)	4.3(17)	3.4(18)	1.7(16)
Coeff. of Variation									
95% Conf. Int.				4.1-4.8	1.7-3.1	-	3.9-4.7	3.0-4.0	1.3-2.1
Range				3.5-5.5	1.3-4.1	<1.3	3.0-6.0	1.8-5.4	1.3-3.4

TABLE A-137. BACTERIAL REDUCTIONS BY CHLORINATION - %

Parameter	Site E				Site H	
	High Flow		Low Flow			
	Sand Filter-Chlorination	Septic Tank-Sand Filter-Chlorination	Sand Filter-Chlorination	Septic Tank-Sand Filter-Chlorination	Sand Filter-Chlorination	Aerobic Unit Sand Filter-Chlorination
Total Bacteria	37	95	98.4	99.94	93.7	98.7
Total Coliforms	98.7	99.99	99.7	99.9996	99.8	99.98
Fecal Coliforms	98.7	99.99	99.7	99.9996	99.6	99.96
Fecal Strep	93.6	99.6	96.8	99.94	98.4	99.7
_Ps_. _aeruginosa_	-	-	92.1	>99.94	98.0	99.7

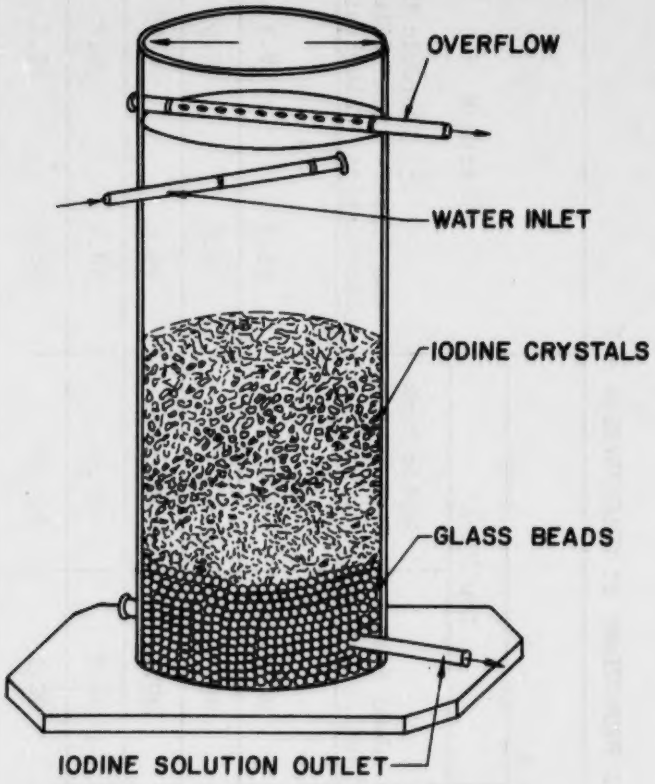


Figure A-67. Iodine saturator.

costs for small installations. If one were to assume that a dosage of 5-10 mg/L would adequately reduce the levels of bacteria in sand filter effluent (Budde, et al., 1977) and an average household flow of 0.76 cubic meters per day, then the yearly chemical cost of iodine would be about $8 to $17 per year.

Ozone--

Like chlorine and iodine, ozone acts to oxidize cell components, thus activating them. It is such a powerful oxidant that it is also effective at reducing organic compounds and taste and odor problems. Because it is an unstable compound, ozone must be produced at the point of application. The primary disadvantage for its use is the expense of reliable equipment needed for its production.

Some of the advantages of ozone are: 1) it is not sensitive to pH, 2) it does not produce a toxic residual, and 3) it is effective at low concentrations. A disadvantage of ozone is that it is fairly sensitive to changes in effluent quality. A sand filtered effluent should be fairly easy to disinfect since much organic material is removed in the process. Ozonation is also quite sensitive to temperature changes.

Utraviolet Light--

The germicidal effectiveness of ultraviolet light has long been known. Only recently has its use been applied to wastewater disinfection. UV is germicidal in the wavelength range of 230 to 300 nm with the optimum efficiency at wavelength 254 nm. Mercury vapor lamps emit a major percentage of their energy at this wavelength, making them the most efficient for use.

Ultraviolet irradiation has been demonstrated to be effective against many types of microorganisms. The primary mode of action is the denaturation of nucleic acids, which makes it very effective against viruses. The primary interferences with UV are turbidity and dissolved substances which absorb UV light. Some of the advantages of UV are: 1) no toxic residuals are formed, 2) destructive action is rapid, 3) no chemical handling required, and 4) automation is easily accomplished. The primary disadvantage of ultraviolet is that the equipment for its application is expensive; however, power costs for small units have been shown to be very low. Because it contains little turbidity, sand filter effluent should be easily disinfected by ultraviolet radiation.

A commercially available ultraviolet water purifier was installed at Laboratory site N where it was used to disinfect aerobic unit effluent and later at site J where it was used to disinfect sand filter effluent. The unit, which is designed for water supply disinfection, consisted of a 75 cm (30 in), low pressure mercury vapor lamp enclosed in a 2.4 cm o.d. quartz jacket and centered in a 7.3 cm i.d. stainless steel tube. The flow rate through the unit varied from 7.5 to 15 liters per minute (2 to 4 gpm) giving a detention time ranging from 11 to 22 seconds. Finally, the ultraviolet energy being emitted by the lamp was 15 W according to the manufacturer's specifications.

A summary of the laboratory data is presented in Table A-138 and a plot of this data is shown in Figure A-68. It is interesting to note that all the bacterial indicators monitored at site N were reduced by about the same order of magnitude ($10^{-3.3}$) or about 99.96%. Separate runs were made at site N to relate flow to bacterial kill (Figure A-69). It was expected that the level of kill would be inversely proportioned to flow (i.e. as flow increased the level of kill should decrease) since the exposure is inversely related to flow. Figure A-69, however, does not indicate such a relationship. An attempt to relate kill to the concentration of suspended solids also indicated no such relationship existed (Figure A-70).

At field site J, the unit was first used to disinfect aerobic unit-sand filter effluent and then septic tank-sand filtered effluent. The flow rate was set at 15 liters per minute (4 gpm). The results of this field test are presented in Table A-139. The reductions of indicator bacteria by the septic tank-sand filter irradiation system were very high about (99.999%) for all the monitored organisms. The reductions were not as high for the aerobic unit-sand filter-irradiation system but it must be noted that the influent organism counts were substantially lower and that the numbers coming out of the ultraviolet unit were usually below the bacterial detection limit (about 1 per 100 ml).

To test the effectiveness of the ultraviolet unit against virus, a 15 liter batch of septic tank-sand filter effluent was inoculated with poliovirus-1

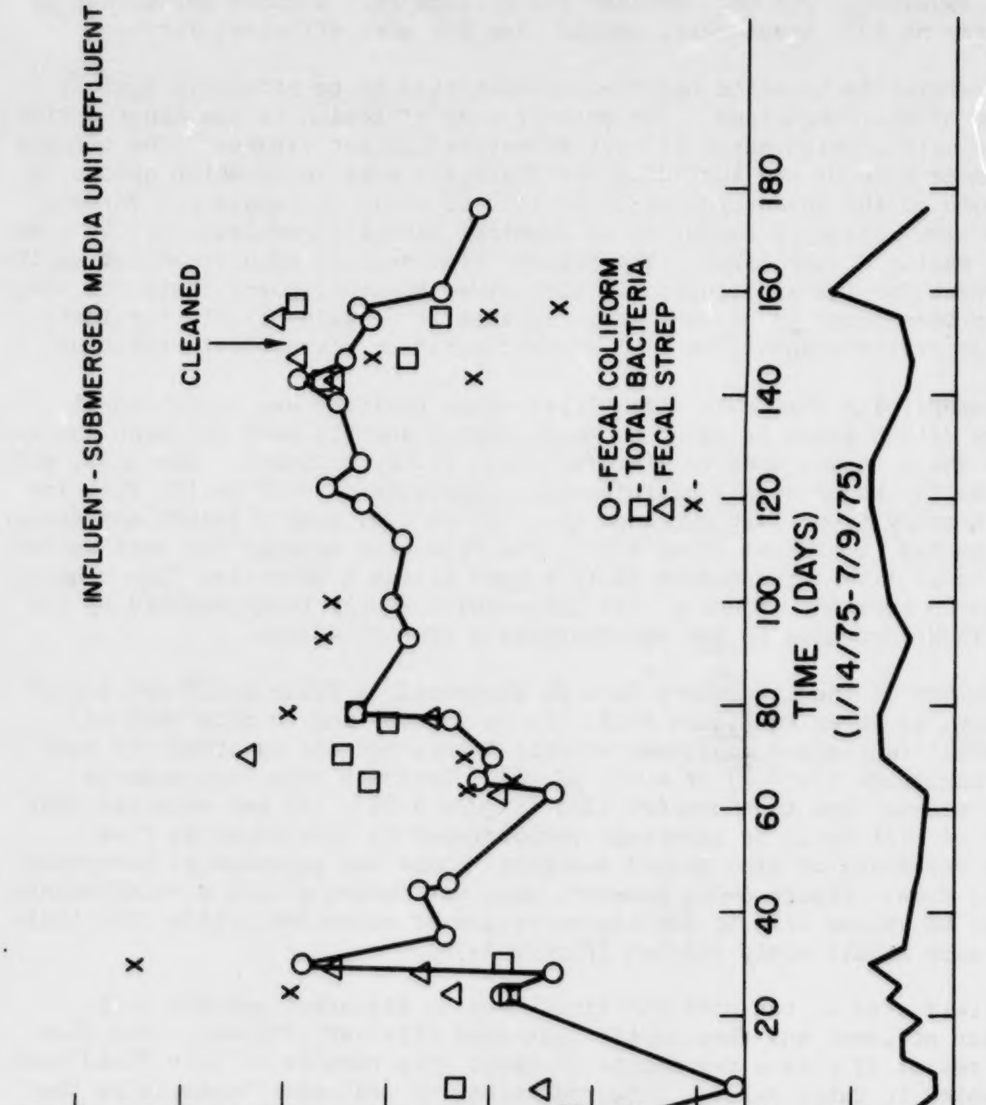


Figure A-68. Bacterial kill by ultraviolet light - lab site N.

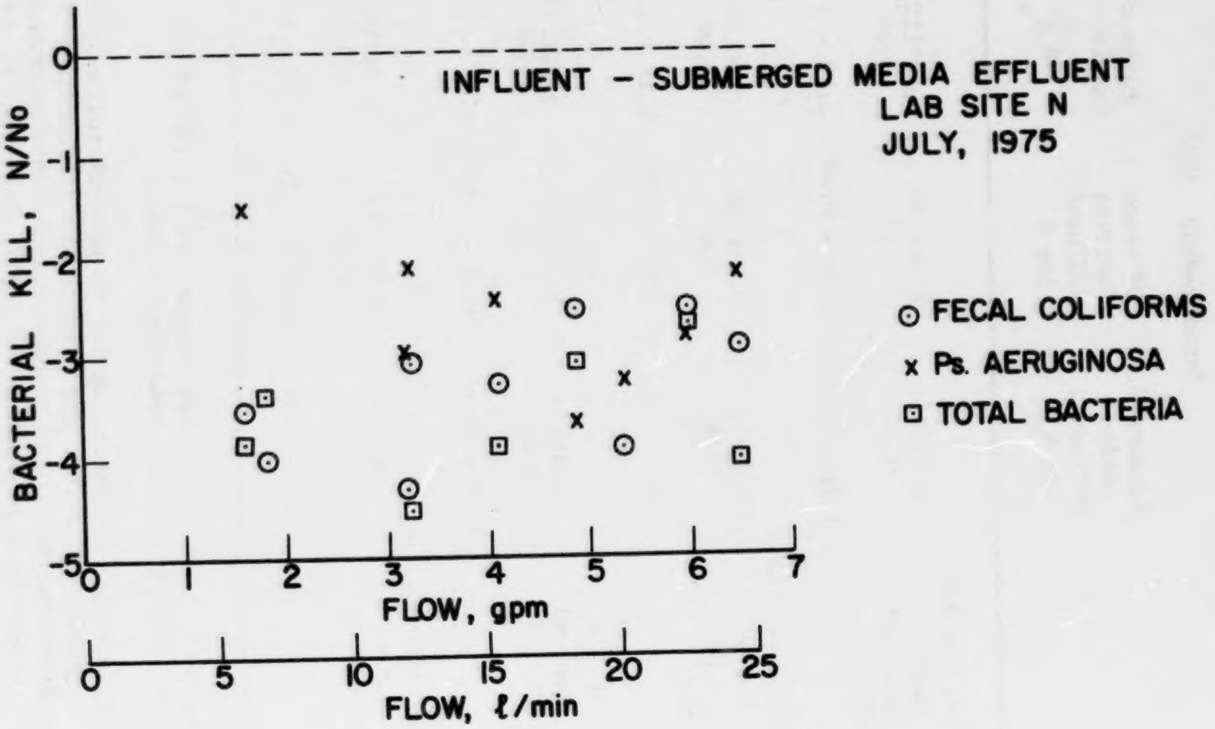


Figure A-69. The effect of flow on the reduction of bacteria through ultraviolet water purifier.

TABLE A-138. BACTERIA REDUCTIONS BY ULTRAVIOLET IRRADIATION - LABORATORY

Site Parameter	Lab Site N January-July, 1975		
	Submerged Media Effluent, log N_o	UV Water Purifier Effluent log N	Ratio of Survivors, $-\log \frac{N}{N_o}$
Total Bacteria, log #/L			
Mean (# of Samples)	9.85(12)	6.58(12)	3.27(12)
Coeff. of Variation	0.04	0.15	0.25
95% Conf. Int.			
Range	9.37 - 10.46	4.93 - 8.07	2.13 - 4.14
Fecal Coliforms, log #/L			
Mean (# of Samples)	5.85(28)	2.45(28)	3.40(28)
Coeff. of Variation	0.14	0.60	0.29
95% Conf. Int.			
Range	4.52 - 7.0	.57 - 3.78	2.16 - 6.40
Fecal Strep, log #/L			
Mean (# of Samples)	5.01(12)	1.70(12)	3.31(12)
Coeff. of Variation	0.14	1.42	0.38
95% Conf. Int.			
Range	4.46 - 6.33	0.30 - 3.90	1.67 - 4.14
Ps. aeruginosa, log #/L			
Mean (# of Samples)	5.26(14)	1.94(14)	3.32(14)
Coeff. of Variation	0.21	0.90	0.40
95% Conf. Int.			
Range	4.11 - 7.4	1.30 - 3.73	0.57 - 6.08

and passed through the unit. The inoculated sample had 4×10^4 PFU/mL, but no virus (< 1 PFU/mL), was detected in the irradiated effluent.

The UV unit had an automatic cleaning device to prevent build-up of residues on the quartz jacket. However, the cleaner was deliberately disconnected in order to determine the consequences of inadequate maintenance. Several months later, another pass of inoculated septic tank-sand filter effluent (> 10^6 PFU of poliovirus-1 per milliter) was put through the unit. No reduction of virus titer was observed. A substantial build-up of opaque material was noted on the quartz jacket. Given proper maintenance, ultraviolet irradiation appears to be capable of achieving good disinfection both for virus and bacteria.

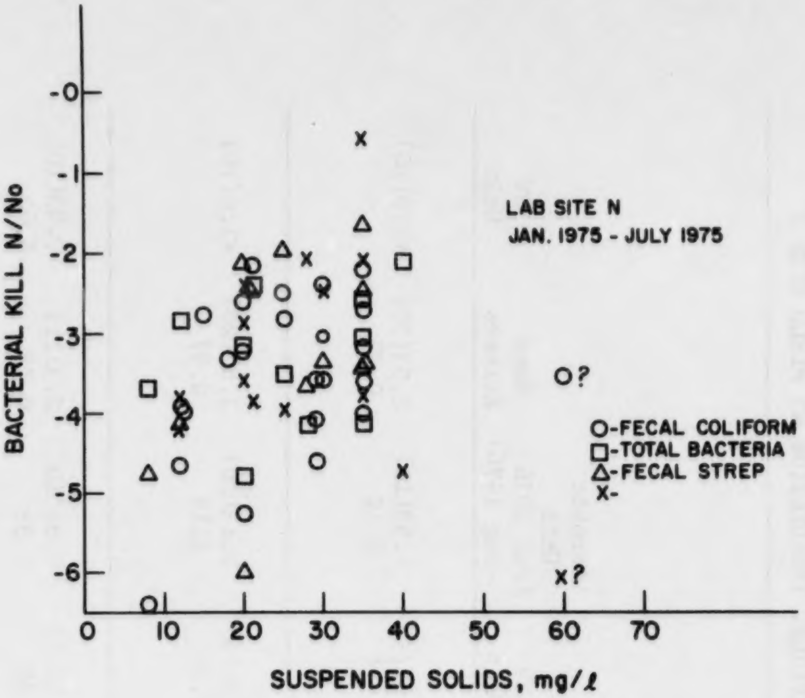


Figure A-70. The effect of influent solids on bacterial reduction by ultraviolet light.

Operation and maintenance of this unit involved periodic cleaning of the quartz enclosure and the provision of power to the UV lamp. Even when the automatic wiping device was operational, the quartz had to be cleaned at least once every 6 months. During the last cleaning, the enclosure was broken and had to be replaced. The lamp, which was operated for over a year, never needed replacement. Although the unit was equipped with an intensity meter, it was not sensitive enough to determine if the lamp was decreasing its output of energy. The ultraviolet lamp utilized power at a rate of 1.5 kwhr per day when operated continuously. When operated intermittently (about 70 minutes per day) the unit only used 2.2 kwhr per month.

Summary--

Although only four disinfectants have been discussed in this section, it is not implied that these are the only practical disinfectants for small flows. Much work has been done demonstrating the effectiveness of bromine as a disinfectant; however, its use is not widespread. Its advantages and disadvantages very much parallel those of chlorine.

TABLE A-139. BACTERIAL REDUCTIONS BY ULTRAVIOLET IRRADIATION AT FIELD SITE J

SITE/DATE PARAMETER	Septic Tank (Sept 1976 -Nov 1976)	Sand Filters	UV Unit	Aerobic Unit (Jul 1976 -Aug 1976)	Sand Filters	UV Unit
Total Coliforms, log #/L						
Mean (# of Samples)	6.73(5)	4.07(8)	1.01(6)	4.53(10)	2.53(20)	<1.0(10)
Coeff. of Variation	0.11	0.13	64.8	0.10	0.37	-
95% Conf. Int.						
Range						
Fecal Coliforms, log #/L						
Mean (# of Samples)	6.38(5)	3.94(8)	0.89(6)	4.13(10)	1.88(20)	<1.0(10)
Coeff. of Variation	0.06	0.13	6.65	0.17	0.57	-
95% Conf. Int.						
Range						
Fecal Strep, log #/L						
Mean (# of Samples)	5.73(5)	3.56(8)	0.79(6)	3.01(10)	2.31(2)	0.83(10)
Coeff. of Variation	0.05	0.24	4.38	0.33	0.73	6.62
95% Conf. Int.						
Range						

Silver, mercury and copper have been shown to be effective disinfectants in low concentrations but concern over toxic effects has limited their use.

Recent studies have concentrated on the use of a combination of disinfectants, one disinfectant covering the disadvantages of the other. Halogen mixtures and the combined use of ozone and chlorine are two possibilities for future use.

At the present time, it appears that the use of hypochlorite, iodine and ultraviolet light are the most promising disinfectants for small flow use. More work is needed to adapt these processes to small flows.

DISINFECTION OF SEPTAGE OR SEPTIC SLUDGE

The agents commonly used for disinfection of water and wastewater include the halogens, ozone and ultraviolet irradiation, but all have serious limitations when applied to concentrated wastes. In fact, disinfection of heavy organic and particulate wastes has long been an unsolved problem.

Concentrated wastes require drastic treatment to provide disinfection and comparatively little work has been done with septage, septic tank sludge or municipal sludge. Those concepts that have been suggested and tested will be mentioned. Although data for the effectiveness of the first groups of disinfection processes are inadequate, the literature should at least be recognized here.

Hess and Breer (1975) reported that pasteurization of municipal sludge at 70° C for 30 min resulted in less than 10 enterobacteriacae/g in 98 to 100% of the samples. Noack and Burger (1974) patented a process where fresh sludge was pasteurized at 70° C for 20 min, followed by mesophilic digestion for 12 days. Pasteurization is, of course, an energy consuming process, so the Noack and Burger patent is attractive for treating municipal sludge from the standpoint that the heated sludge is then subjected to a mesophilic or thermophilic digestion so the heat from pasteurization can be conserved and provide further benefit.

Gamma irradiation has been tested as another approach to sludge sterilization (Suess, 1974; Herruhut and Bosshard, 1974; Hess and Breer, 1975). Hess and Breer (1975) indicated that 300 Krad resulted in less than 10 enterobacteriacae/g in 97.2% of the samples.

Pasteurization and irradiation are processes that require specialized equipment and technical personnel. The complexity and cost of the processes may account for infrequent use.

There are a number of simpler methods for disinfecting concentrated wastes. Feige, et al., (1975) studied lime stabilization of septage followed by sand-bed dewatering. They found that liming sludge to a pH of 11.5 produced fecal coliform (FC) reduction within the first day to levels below 1 x 10^3/100 mL (at least 3 logs of reduction). The fecal streptococci (FS) were not killed as readily, their counts being lowered no more than 1 log within the first day of pH 11.5 treatment, but numbers dropped about 3 logs after 5 days. Similiar results were found in lime stabilization of municipal sludge (Farrell, 1974; Doyle, 1967). Salmonellae were initially present in the municipal sludge and testing for them after lime treatment revealed the pH of 11.5 to 12 was lethal to them.

Sobsey, et al., (1974) examined a variety of chemicals for the disinfection of holding tank sewage. Included were calcium hypochlorite, zinc sulfate, phenol, formaldehyde, methylene blue, benzalkonium chloride and cetylpyridinium chloride. They found that formaldehyde, methylene blue, benzalkonium chloride and cetylpyridinium chloride were effective bactericides and virucides at pH 10.5, but not at lower pH values. Calcium hypochlorite, zinc sulfate and phenol were ineffective when used on holding tank sewage.

The Feige, et al., (1975) and Sobsey, et al., (1974) findings provide at least a basis for considering disinfectants for septage. Lime stabilization appears capable of killing both fecal indicators and pathogens, but long contact times (days) and the high pH of the resulting sludge would be two disadvantages of the process. The agents which were successful in killing bacteria and inactivating viruses in the Sobsey study with holding tank sewage deserve further consideration for septage since the two wastes are both high in organic matter and solids.

Also noteworthy is the approach which combines alkalinity with a disinfectant to improve performance. This approach may also improve disinfection of other agents.

Materials and Methods

A number of disinfectants were tested on septic tank sludge and the course of these experiments (Deininger, 1977). The method of analysis for gluteraldehyde was based upon the reactive index of gluteraldehyde which J. H. Luft (1976) (University of Washington, personal communication) reported rises above that of water (N_D^{20} 1.3330) by 0.0168 for each 10% of gluteraldehyde content. No analytical method was found for measuring gluteraldehyde rendered in sludges.

Bacterial analyses were performed in accordance with methods described in Appendix C.

Septic tank sludges that were tested included those from site N, and a farm home occupied by two adults (LD). The septage from site N had a total solids concentration of approximately 34,000 mg/L (after sieving to remove gross solids) whereas the farm home septage solids were about 70,000 mg/L.

Testing of Five Selected Disinfectants on Sludge

A variety of agents was selected for testing of their ability to kill fecal coliforms (FC) in sludge. Elemental iodine, like chlorine has been used to disinfect drinking water and wastewater. It was expected that sludge organic matter would lower its effectiveness, but it was included in the study for comparative purposes. Iodophor compounds are commonly used for surgical scrubbing and cleaning dairy equipment. The iodine is coupled to carrier molecules which allows iodine to be released slowly, thereby reactive iodine remains in solution for longer periods of time. Benzalkonium chloride (benzalkonium-Cl) belongs to the class of cationic detergents which are widely used to clean and disinfect surfaces, particularly in hospitals and laboratories. Formaldehyde has been found to be an effective disinfecting agent for both bacteria and viruses, if acting at pH 10.5. Glutaraldehyde was included in the study because of its chemical similarity to formaldehyde, its successful use as a sterilizer of surgical equipment (Stonehill, et al., 1963), and its well known properties as a biological fixative (Hayat, 1970).

These agents were tested on site N septage over a range of concentrations considered reasonable from literature findings. The pH values at which the agents were tested were based on recommendations from the literature. Benzalkonium-Cl and formaldehyde were reported most effective at pH 10.5 in treating holding tank sewage. It was the experience of this study that FC kills were very different at pH 10 and 11, and that pH 10.5 was near to the pH where bacterial death took place with no additional disinfecting agent. For this reason pH values of 9 and 10 were chosen as maximum test levels to avoid any masking effect of pH being the dominant factor in FC kills, and also to minimize caustic handling problems. Contact times of 1, 2 and 4 hrs were allowed for all of the agents to facilitate comparisons.

Results of the experiment are found in Table A-140. The data are expressed as percent survivors to simplify comparisons between agents tested. An asterisk beside a survivor percentage indicates at what percentage survivors the FC counts were less than 200/100 mL.

Elemental iodine and iodophor were most effective at initial dosages of 500 mg/L iodine. Iodine was only tested at pH 7.3 since that is near the optimum pH for iodine disinfection (Chang, 1958). Also, this is within the normal pH range for most septages.

Benzalkonium-Cl was bactericidal at concentrations of 500 mg/L and somewhat at 300 mg/L. The bactericidal activity does appear to be enhanced by increasing the pH from 9 to 10.

Formaldehyde treatment at pH 10 and glutaraldehyde treatment at pH 9 and 10 gave the best performance of all of the agents tested in this experiment. A concentration of 200 mg/L of formaldehyde reduced FC below 200/100 mL within 4 hrs of contact. There seemed to be no advantage in combining benzalkonium-Cl and formaldehyde for sludge treatment. Glutaraldehyde produced FC kills to levels below 200/100 mL even at the 100 mg/L dosage. The kill was rapid with the majority of deaths taking place during the first hr of contact. On the basis of these preliminary experiments, formaldehyde and glutaraldehyde seem to warrant further study.

Formaldehyde Studies

Further detailed work was performed with formaldehyde to better characterize its effectiveness on septic tank sludges. Results of experiments conducted

TABLE A-140. TESTS OF VARIOUS DISINFECTING AGENTS AGAINST FC IN SITE N SEPTAGE*(Deininger, 1977)

			Percent FC survivors after contact time (hr) of:		
Disinfectant	mg/L	(FC/100 mL)	1	2	4
$KI + I_2$	100	4.3×10^6	100	100	100
pH 7.3	300	4.3×10^6	100	100	100
	500	4.7×10^6	0.0020	0.00050	0.0050
Iodophor	100	9.3×10^6	26	100	46
pH 7.3	300	9.3×10^6	100	0.025	0.42
	500	4.3×10^6	0.10	0.17	0.053
Benzalkonium-Cl	100	9.3×10^6	46	460	460
pH 9.0	300	2.3×10^6	100	400	100
	500	4.3×10^6	1.0	< 0.00070**	0.00093**
Benzalkonium-Cl	100	4.3×10^6	100	100	53
pH 10.0	300	9.3×10^6	4.6	0.081	0.0025
	500	7.5×10^6	0.0057	0.00053**	0.0031
Formaldehyde	50	1.5×10^7	2.9	5.0	29
pH 9.0	100	7.5×10^6	10	10	5.7
	200	1.5×10^7	29	0.62	2.9
Formaldehyde	50	9.3×10^6	81	2.5	0.25
pH 10.0	100	4.3×10^6	10	0.053	0.053
	200	7.5×10^6	0.31	0.0053**	0.00067**
Formaldehyde/	50/100†	4.3×10^6	53	100	56
benzalkonium-Cl	100/200	1.5×10^7	29	2.9	2.9
pH 9.0	200/300	1.5×10^7	15	2.9	6.2
Formaldehyde/	50/100	4.3×10^6	5.3	100	10
benzalkonium-Cl	100/200	3.9×10^6	59	38	3.8
pH 10.0	200/300	9.3×10^6	2.5	0.0046	< 0.00032**
Glutaraldehyde	100	4.3×10^6	< 0.070	0.0070	<0.00070**
pH 9.0	200	1.5×10^7	< 0.020	0.0020	0.00020**
	300	9.3×10^6	0.0032	< 0.00032**	0.00032**
Glutaraldehyde	100	4.3×10^6	< 0.0070	< 0.00070**	< 0.00070**
pH 10.0	200	4.3×10^6	< 0.0070	< 0.00070**	< 0.00070**
	300	2.3×10^6	0.013	< 0.0013**	< 0.0013**

* SSWMP sludge, TS = 6,000 mg/L

** Less than 200 FC/100 mL surviving treatment

† Formaldehyde = 50 mg/L
benzalkonium-Cl = 100 mg/L

with the farm home (LD) septage (TS = 70,000 mg/L) appear in Table A-141. Of the groups tested, FC and _P. aeruginosa_ were most reactive to formaldehyde at pH 10.0. It is apparent from the control data that formaldehyde, not pH alone accounted for the bacterial kill measured.

The effect of sludge concentration on required formaldehyde dose is shown in Table A-142, again employing (LD) septage. A more rapid rate of killing is apparent at the lower sludge concentrations. At 12 hours of contact, however, approximately the same reduction of FC was achieved. In additional experiments, it was shown that allowing sufficient contact time (6 to 12 hrs) was more important in providing adequate bacterial kills than was vigorous mixing.

Gluteraldehyde Studies

Bacterial Disinfection--

Further testing was also performed with gluteraldehyde. Results of experiments conducted with the (LD) septage using gluteraldehyde at pH 7.1 appear in Table A-143. The FC and P. aeruginosa bacterial groups were the most sensitive tested, and a dose of 750 mg/L or greater reduced FC to undetectable levels (7 log drop) within 3 hours. The total bacterial counts (TBC) were little effected by even the highest doses of gluteraldehyde. Further analyses suggested that spores accounted for the resistance to this disinfectant.

The effect of sludge concentration on gluteraldehyde disinfection at pH 7.1 appears in Table A-144. As with formaldehyde, it was apparent that the same dose of disinfectant was more effective at lower solids concentrations.

Since the binding capacity of glutaraldehyde to target chemical groups increases with increased alkalinity (Hayat, 1970), it would be expected that this disinfectant would be most useful at pH values in excess of 7.0. The effect of septage pH on disinfection is shown in Table A-145. It is apparent that gluteraldehyde treatment at pH 7 or lower results in greater FC and FS reduction than treatment in the weakly alkaline range. This is likely due to the non-specific binding of aldehyde to inert protein within the sludge. At lower pH, more effective penetration is achieved, even though the rate of binding is decreased.

Studies in the temperature range of 6 to 20° C indicate that within this range there is little effect on glutaraldehyde disinfection at pH 7.1.

Virus Inactivation--

The procedure by which the model septage suspensions were inoculated with virus was as follows:

1. Septage (LD sludge, TS = 70,000 mg/L) was sieved through 1 cm (3/8 in) hardware cloth and blended at "high" speed with a Waring blender. The sludge thus prepared characteristically has a TS concentration of 66,000 mg/L.

TABLE A-141. FORMALDEHYDE TREATMENT OF LD SEPTAGE AT 20° C AND pH 10.0* (Deininger, 1977)

Formaldehyde Concentration	Bacterial group**	Bacterial survivors after contact of: 1 hr	6 hr	12 hr
Control pH 7	FC/100 mL	1.7×10^8	1.7×10^8	1.6×10^9
	FS/100 mL	6.6×10^6	5.0×10^6	5.5×10^6
	PA/100 mL	No data	2.3×10^4	2.3×10^4
	TBC/mL	3.2×10^6	8.4×10^6	1.5×10^7
Control pH 10	FC/100 mL	1.7×10^8	7.0×10^7	9.2×10^8
	FS/100 mL	6.4×10^6	9.0×10^6	7.4×10^6
	PA/100 mL	No data	7.0×10^4	1.3×10^3
	TBC/mL	2.4×10^6	4.4×10^6	1.1×10^7
500 mg/L	FC/100 mL	1.7×10^7	3.5×10^4	$<2.0 \times 10^1$
	FS/100 mL	5.8×10^6	3.5×10^6	2.3×10^6
	PS/100 mL	5.0×10^1	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	TBC/mL	1.1×10^6	1.3×10^5	1.7×10^5
750 mg/L	FC/100 mL	3.5×10^6	4.9×10^2	$<2.0 \times 10^1$
	FS/100 mL	7.6×10^6	9.1×10^5	3.2×10^5
	PA/100 mL	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	TBC/mL	7.2×10^5	1.7×10^5	1.2×10^5
1,000 mg/L	FC/100 mL	1.3×10^6	2.2×10^2	$<2.0 \times 10^1$
	FS/100 mL	5.9×10^6	6.9×10^5	5.0×10^2
	PA/100 mL	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	TBC/mL	1.7×10^5	1.5×10^5	1.1×10^5
1,250 mg/L	FC/100 mL	7.0×10^4	2.0×10^1	$<2.0 \times 10^1$
	FS/100 mL	5.0×10^6	7.0×10^4	$<1.0 \times 10^3$
	PA/100 mL	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	TBC/mL	1.8×10^5	1.2×10^5	1.5×10^5

* LD Sludge, TS - 70,000 mg/L

** Initial numbers

FC/100 mL = 2.4×10^8
FS/100 mL = 8.1×10^6
PA/100 mL = 3.5×10^4
TBC/mL = 6.8×10^6

TABLE A-142. INFLUENCE OF SLUDGE SOLIDS CONCENTRATION ON FORMALDEHYDE DISINFECTION* -pH 10 (Deininger, 1977)

Total Solids mg/L	Formaldehyde dose mg/L	Bacterial group	Initial Bacteria/ 100 mL	Percent FC and FS survivors at following contact times (hr):		
				1	2	3
52,500*	500	FC	1.3×10^8	1.0	0.001	0.000015**
		FS	3.8×10^6	95	63	58
	1,000	FC	3.5×10^8	0.0037	0.0000057**	0.0000057**
		FS	5.2×10^6	85	5.8	0.0019
35,000	500	FC	3.3×10^7	0.061	0.00052**	0.000061**
		FS	2.8×10^6	6.4	2.9	7.9
	1,000	FC	7.0×10^7	0.029	0.000029**	0.000029**
		FS	3.4×10^6	5.9	0.18	0.0029
17,500	500	FC	7.9×10^7	0.028	0.000025**	0.000025**
		FS	1.8×10^6	19	2.5	0.67
	1,000	FC	4.6×10^7	0.00025**	0.000025**	0.000025**
		FS	1.6×10^6	7.5	0.063	0.0063

* LD sludge, TS = 70,000 mg/L before dilution (20°C).
** Indicates < 200 FC/100 mL.

TABLE A-143. GLUTARALDEHYDE TREATMENT OF LD SEPTAGE* (Deininger, 1977)

Glutaraldehyde concentration	Bacterial group**	Bacterial survivors after contact of: 1 hr	3 hr	6 hr	12 hr
500 mg/L	FC/100 mL	$<2.0 \times 10^3$	8.0×10^1	2.0×10^1	2.2×10^3
	FS/100 mL	2.0×10^5	1.0×10^4	4.0×10^3	1.5×10^6
	PA/100 mL	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	TBC/mL	2.9×10^5	1.7×10^5	1.7×10^5	2.0×10^5
750 mg/L	FC/100 mL	5.0×10^2	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	FS/100 mL	1.0×10^4	$<1.0 \times 10^2$	$<1.0 \times 10^2$	$<1.0 \times 10^2$
	PA/100 mL	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	TBC/mL	2.8×10^5	1.2×10^5	1.4×10^5	1.5×10^5
1,000 mg/L	FC/100 mL	2.0×10^1	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	FS/100 mL	5.0×10^3	$<1.0 \times 10^2$	$<1.0 \times 10^2$	$<1.0 \times 10^2$
	PA/100 mL	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	TBC/mL	3.2×10^5	1.7×10^5	1.2×10^5	1.4×10^5
1,250 mg/L	FC/100 mL	1.7×10^3	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	FS/100 mL	2.0×10^3	$<1.0 \times 10^2$	$<1.0 \times 10^2$	$<1.0 \times 10^2$
	PA/100 mL	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$	$<2.0 \times 10^1$
	TBC/mL	3.0×10^5	1.4×10^5	1.3×10^5	1.4×10^5

* LD sludge, TS = 70,000 mg/L; pH = 7.1; T = 20° C

** Initial numbers

FC/100 mL = 1.3×10^8
FS/100 mL = 1.8×10^6
PA/100 mL = 2.2×10^3
TBC/mL = 2.9×10^6

TABLE A-144. INFLUENCE OF SOLIDS CONCENTRATION ON GLUTARALDEHYDE DISINFECTION* (500 mg/L TREATMENT) (Deininger, 1977)

Sludge Solids (mg/L)	Bacterial group	Initial number per 100 mL	Percent survivors after contact of:					
			1 hr		3 hr		6 hr	
			average	range	average	range	average	range
52,000*	FC	3.5×10^7	0.049	0.026 - 0.069	0.037	0.0049 - 0.069	0.034	0.00023 - 0.069
	FS	5.0×10^7	0.068	0.048 - 0.088	0.36	0.016 - 0.70	0.080	0.0072 - 0.15
35,000	FC	9.2×10^7	0.0039	0.0019 - 0.0059	0.0012	0.00053 - 0.0018	0.0013	0.0012 - 0.0014
	FS	1.8×10^7	0.052	0.043 - 0.061	0.030	0.052 - 0.0072	0.027	0.019 - 0.034
17,500	FC	3.5×10^7	0.00013**	0.000057** - 0.00020**	< 0.000057**	0	0.000094**	0
	FS	1.2×10^7	0.010	0.0058 - 0.014	< 0.00083	0	0.0040	0.00083 - 0.0071

* LD sludge, TS = 70,000 mg/L before dilution, T = 20°C, pH = 7.1
** Indicates 200 FC/100 mL

TABLE A-145. INFLUENCE OF SLUDGE pH ON GLUTARALDEHYDE DISINFECTION OF BACTERIA* (Deininger, 1977)

pH Initial/Final	Glutaraldehyde dose (mg/L)	Bacterial group	Initial counts	Percent survivors** after contact of: 1 hr	3 hr	6 hr
8.5/8.3	Control	FC	9.2×10^8	-	-	59
		FS	7.2×10^7			110
	500	FC	-	0.049	0.013	0.048
		FS	-	0.91		1.0
	1,000	FC	-	0.071	0.015	0.0026
		FS	-	0.14	0.12	0.15
7.1/7.1	Control	FC	1.7×10^7	-	-	42
		FS	2.4×10^7	-	-	96
	500	FC	-	0.0053	0.015	0.0076
		FS	-	0.042	0.14	0.058
	1,000	FC	-	0.038	0.0038	0.014
		FS	-	0.021	0.0083	<0.0042
6.0/5.9	Control	FC	9.0×10^7	-	-	No data
		FS	4.0×10^7	-	-	90
	500	FC	-	0.017	0.019	0.00020†
		FS	-	0.44	0.0040	<0.0020
	1,000	FC	-	0.014	0.0013	0.00011†
		FS	-	0.044	0.012	0.0080
5.4/5.8	Control	FC	9.0×10^7	-	-	No data
		FS	4.0×10^7	-	-	90
	500	FC	-	0.0022	0.014	0.00030
		FS	-	0.96	0.026	0.022
	1,000	FC	-	<0.27	0.00016†	0.000094†
		FS	-	0.0020	0.066	0.0024

* LD sludge, TS = 70,000 mg/L; T = 20° C

** Average of duplicate tests
Temperature - 20° C

† Indicates <200 FC/mL

2. A measured volume of septage sufficient for an experiment (usually between 200 to 400 mL) was stirred in a sterile 500 mL erlenmeyer flask with a magnetic stirring bar.

3. The poliovirus stock at 3.0 x 10^9 PFU was added at the rate of 0.5 mL per 100 mL septage with constant stirring for 30 min. Final virus concentration = 1.5 x 10^7 FPU/mL.

4. Virus-inoculated septage was then refrigerated at 4° C overnight to allow adsorption of virus to sludge particles.

5. The next day the inoculated septage was again stirred for 15 min to mix and resuspend particles. The virus-inoculated septage was then ready for use in experiments.

It was found during the bacterial studies that glutaraldehyde treatment of neutral or slightly acidic septage resulted in greater bacterial kills than in alkaline sludge. Consequently, it was necessary to examine whether pH would play a role in virus inactivation when septage containing virus was treated with glutaraldehyde.

Following the above procedure, the stirred virus-inoculated septage was mixed for 15 min with a magnetic stirring bar after which the material was divided into sterile flasks of 100 mL each. The contents of the flasks were adjusted to pH 5.0, 6.0, 7.1 and 8.0, respectively, with 10% KOH or 10% HCl. Thirty milliliter quantities of septage, two each for each pH point, were added to sterile 25 x 200 mm test tubes and equilibrated to 20° C. Each tube was treated with 500 mg/L glutaraldehyde and allowed 3 hrs of contact before it was assayed for virus. A control was run at pH 7 without glutaraldehyde treatment to determine the initial level of PFU in the septage. The recovery procedure described earlier was followed for all samples and included FCS addition, adjustment to pH 9 and sonication. The plaque assay was performed on HeLa cells according to the usual procedure.

Table A-146 shows that pH from 5.0 to 8.5 had little effect on the number of PFU found in the virus-septage preparation after treatment with glutaraldehyde. Despite the small differences in virus inactivation with glutaraldehyde at the four pH values, slightly greater inactivation was found in the acidic range. The close agreement of the duplicate tests at each pH suggest that although the improvement in inactivation under acid conditions was small, it was probably significant rather than due to imprecision of the analyses. This is compatible with the results from bacterial experiments in which glutaraldehyde treatment of septage in the acidic range favored bacterial kill, but it appears that the pH is of only minor importance to viral inactivation.

Since the 500 mg/L glutaraldehyde level applied in this experiment reduced the PFU only 0.8 to 1.5 logs, it would be desirable to provide a higher degree of inactivation. The bacterial work with glutaraldehyde also showed a 500 mg/L dose to be marginally effective in destroying fecal coliform and fecal streptococcal bacteria in septage, but a 1,000 mg/L treatment was more effective. An experiment comparing viral inactivation in septage at pH 7.1 with glutaraldehyde at dose levels of 500 to 1,000 mg/L was then performed.

TABLE A-146. INFLUENCE OF pH ON POLIOVIRUS INACTIVATION IN SEPTAGE* WITH 500 MG/L GLUTARALDEHYDE

pH Initial/Final	Percent virus survivors after 3 hr contact Average	 Range
5.0/4.6	6.6†	6.4 to 6.9
6.0/6.4	4.5	3.6 to 5.3
7.1/7.3	11	0
8.5/7.8	15	14 to 16

†Initial number of poliovirus = 5.8×10^6/mL

*LD sludge, sieved, blended, TS = 66,000 mg/L

Temperature = 20° C

Sieved, blended septage (LD sludge, TS = 66,000 mg/L) was inoculated with poliovirus to a level of 1.5×10^7 PFU/mL, as described before. After overnight adsorption of 4°C, 30 mL quantities of virus-inoculated septage were dispensed into sterile 25 x 200 mm test tubes and equilibrated to 20° C for about 1.5 hrs. Glutaraldehyde was then added to the tubes of septage to concentrations of 500 to 1,000 mg/L. At selected time periods, duplicate tubes of the two dose rates were treated for 1 min with 1,260 mg/L Na_2SO_3. Next, fetal calf serum (FCS) was added to each septage sample, and the septage FCS mixture was adjusted to pH 9. This mixture was then stirred with a stirring bar for 15 min. The remainder of the recovery procedure and plaque assays were performed as usual. Control samples received no glutaraldehyde but were otherwise processed as above.

The 500 mg/L glutaraldehyde level caused 1.4 $\log_{10}$ of reduction of poliovirus titer, The virus inactivation occurred mostly during the first hour of treatment with little change during the 1 to 3 hour span of contact. The dose rate of 1,000 mg/L produced 2.7 $\log_{10}$ of poliovirus inactivation. Inactivation was most rapid during the first 1.5 hours of contact, but nearly 0.5 log of inactivation took place between 1.5 and 3 hours.

Poliovirus appears to be more resistant to glutaraldehyde treatment than the fecal indicator bacteria. A glutaraldehyde dose of 500 mg/L produced approximately 3 and 4 $\log_{10}$ of reduction, respectively, of fecal streptococci and fecal coliforms compared to 1.4 $\log_{10}$ of reduction of poliovirus. A 1,000 mg/L glutaraldehyde dose produced about 4 $\log_{10}$ reduction of fecal streptococcus and 4 to 7 $\log_{10}$ of fecal coliform reduction while poliovirus was reduced approximately 3 $\log_{10}$. Just as a 500 mg/L glutaraldehyde treatment was considered marginal for bacteria, it also appears to be marginal for virus. A 1,000 mg/L concentration of glutaraldehyde is therefore recommended.

The treatment procedure of septage with glutaraldehyde could be performed in the tank of the pumper truck. After estimating the volume of septage to be pumped, the required amount of glutaraldehyde could then be added and be mixed with the septage in one of two ways. First, the glutaraldehyde could be added to the empty truck tank and mixed by pumping the septage into it. A gradient of glutaraldehyde would form as the septage entered and would be diluted to the proper concentration when the contents of the septic tank filled the truck tank. Alternately, the glutaraldehyde could be metered into the septage as it is pumped into the tank of the truck. In either procedure, additional mixing would occur due to sloshing back and forth of the liquid as the truck moved.

As mentioned, 1 to 3 hrs of contact is needed before bacterial kill and viral inactivation is nearly complete (some additional bacterial kill may take place for up to 6 hrs). This could not be feasible if it would have to be held in the truck tank for an entire contact period before discharge. However, if the truck empties its tank contents into a holding tank at the disposal site, additional disinfection would likely occur in that vessel.

GREY WATER TREATMENT

Typically, when segregated systems are suggested with the toilet wastes handled through use of a non-conventional toilet system (e.g., composting, incinerating, recycle, low-volume flush/holding tank) grey water treatment and disposal involves a conventional septic tank-soil absorption system (possibly of reduced size). Although this is an obvious method of treatment and disposal, the characteristics of grey water appear to be such that simple alternative methods might exist which could be more desirable in certain applications. Some treatment methods might even facilitate surface disposal or outside reuse of the grey water. To enhance the near non-existent data base regarding the on-site treatment of household grey water, this preliminary study was conducted. The major objectives established were to:

1. Identify the comparative performance of septic tank treatment of grey water versus that of combined black and grey water,

2. Identify the comparative performance of septic tank treatment of grey water utilizing a 2.0 m^3 (500 gal) septic tank versus a 4.0 m^3 (1000 gal) septic tank,

3. Investigate the comparative performance of intermittent sand filtration of septic tank effluent as generated by grey water versus combined black and grey water, and

4. Identify areas for further research.

To provide a controlled environment for this preliminary study and facilitate the above types of comparisons, this effort was conducted at laboratory site N. This grey water treatment study was initiated at this facility in April, 1976 with active data collection beginning in July, 1976. The results presented and discussed herein involve a one-year period from April, 1976 to April, 1977.

Data Generation Methods

To summarize briefly, at laboratory site B the wastewater generated by typical household events was simulated utilizing conventional appliances as well as specially designed equipment and consumer materials/waste products (more detail is provided elsewhere in this Appendix). Utilizing a sophisticated electronic control system, the simulated household events were made to occur intermittently during the day, yielding a loading pattern representative of that generated by a family of four. By staggering the schedule of events, the same event sequence and daily loading could be applied to each of several treatment units.

The general flow sheet for this study is presented in Figure A-71. As shown, the individual events were simulated and the resulting grey waters were directed to each of two, septic tanks of 2.0 m^3 (500 gal) and 4.0 m^3 (1000 gal) size. A third septic tank of 4.0 m^3 size, received the same grey waters, as well as toilet flush wastewaters. The effluent from each septic tank flowed into its own 0.2 m^3 (55 gal) sump from which it was intermittently pumped. The effluent from the 2.0 m^3 grey water tank sump was

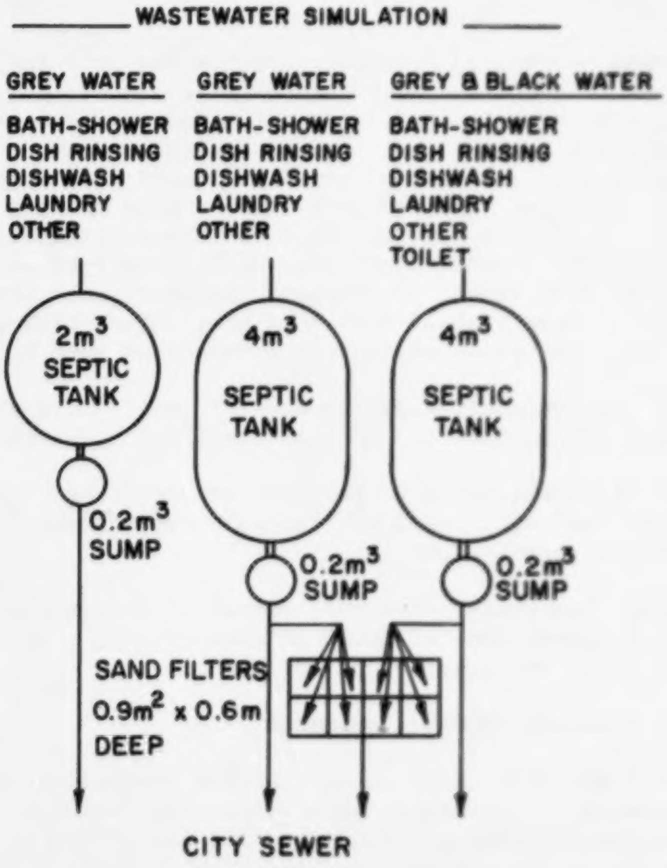

WASTEWATER SIMULATION
GREY WATER
BATH-SHOWER
DISH RINSING
DISHWASH
LAUNDRY
OTHER
GREY WATER
BATH-SHOWER
DISH RINSING
DISHWASH
LAUNDRY
OTHER
GREY & BLACK WATER
BATH-SHOWER
DISH RINSING
DISHWASH
LAUNDRY
OTHER
TOILET
2m³ SEPTIC TANK
4m³ SEPTIC TANK
4m³ SEPTIC TANK
0.2m³ SUMP
0.2m³ SUMP
0.2m³ SUMP
SAND FILTERS 0.9m² x 0.6m DEEP
CITY SEWER

Figure A-71. Grey water treatment study - flow sheet.

discharged directly to the city severage system, as was a majority of the effluent from each of the 4.0 m^3 tank sumps. A portion of the effluent from each of the 4.0 m^3 septic tanks was applied to a series of sand filters (8). Each filter had a surface area of approximately 0.09 m^2 (1.0 ft^2) and a depth of 0.6 m (2.0 ft).

Simulated Raw Wastewater--

Each of the three septic tanks received the same grey waters and one of the units also received toilet flush wastewaters. The day-to-day loading was constant for all events with the exception of the laundry which varied from none to three occurrences per day. A summary of the make-up of the simulated event wastes is shown in Table A-147 with the daily loading pattern to each unit depicted in Figure A-72.

The average characteristics of the simulated wastewaters were identified through periodic analyses of the individual material constituents, the results of which were composited mathematically. Several actual influent composites were also taken and analyzed. Based on these results, the characteristics of each of the two simulated influents were, on an average daily basis, as shown in Table A-148. Variations from the values shown in Table A-148 occurred as a result of the day-to-day laundry variation. Further, infrequent programmer malfunctions and maintenance requirements also resulted in variations from the values shown in Table A-147, as well as Table A-148 and Figure A-72 during the course of the study.

TABLE A-147. SIMULATED WASTEWATER EVENT CHARACTERISTICS

Event Simulated	L / Event	Number / Day	L / Day	Material / Event	Quantity / Day
Bath	79	1	79	Liquid Soap	25 mL
Shower	102	1	102	Liquid Soap	25 mL
Dish Rinse	11	3	33	Food Slurry	780 mL
Dish Wash	53	1	53	Detergent	80 gms
Laundry	180	3	540	Detergent	375 gms
		2	360	Detergent	250 gms
		0	0	-	-
Miscellaneous	19	3	57	-	-
Toilet	19	12	228	Urine Solution	3.84 L
				Hog Manure	1400 gms
				Paper	19 gms

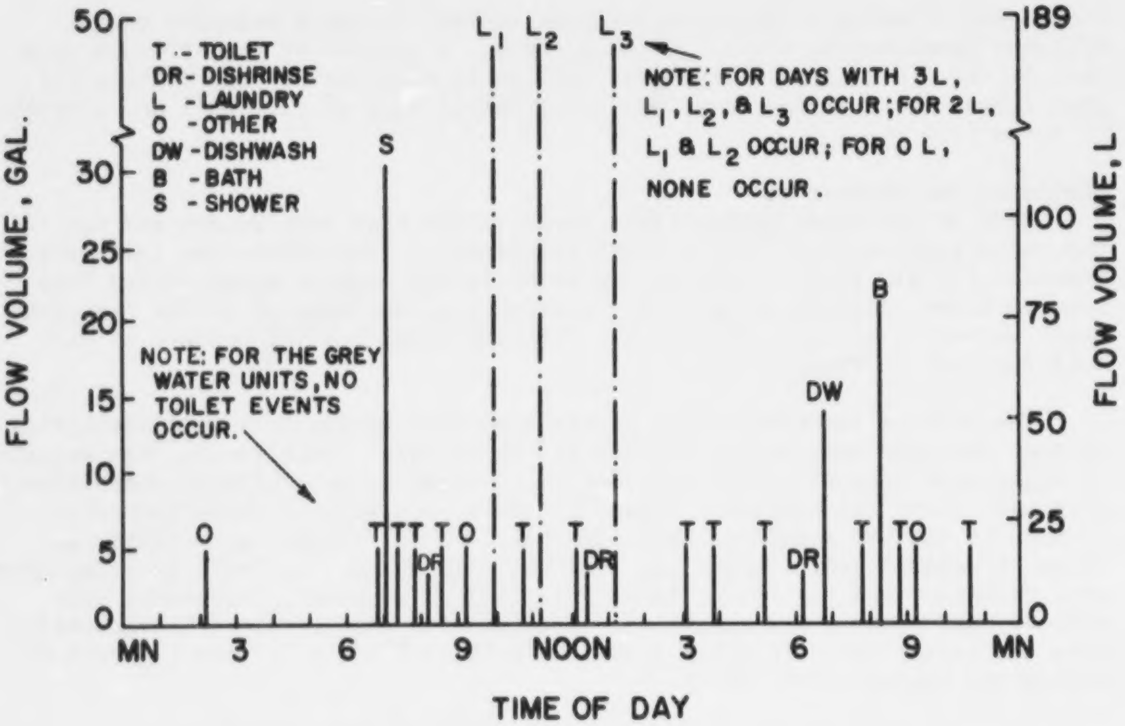


Figure A-72. Approximate daily loading pattern.

TABLE A-148. SIMULATED RAW WASTEWATER CHARACTERISTICS - DAILY AVERAGES*

Parameter	Grey Water		Combined Wastewater	
	Mass/Day	mg/L	Mass/Day	mg/L
Flow	484 Liters	-	711 Liters	
BOD_5	108 Grams	220 mg/L	186 Grams	260 mg/L
COD	203	420	519	730
TS**	396	820	874	1230
TSS	52	110	295	410
TN	5.6	12	57	80
TP	21.5	44	40.2	57

* Does not take into account infrequent down-days, programmer malfunctions, etc.

** Includes 300 mg/L total solids contribution from the tap water

Septic Tank Operation--

All three of the septic tanks utilized in this study had been used during the previous research effort at laboratory site N. All three tanks had received simulated combined wastewater for approximately 15 months prior to the start of this study. The 4.0 m^3 tanks were left as is, while the 2.0 m^3 tank was pumped down and cleaned out except for a few cm of residual material. For three months prior to the start of this evaluation, the three septic tanks were fed their respective new raw wastewaters, grey or combined, to stabilize their performance.

During the course of this study, daily flow composited samples of each septic tank effluent were collected weekly. These samples were obtained, as desired, by directing a fraction of the wastewater pumped from each septic tank effluent sump to refrigerated containers through a 0.6 cm diameter sample line. The samples collected were analyzed for various chemical/physical parameters. In addition to these composite samples, several grab samples were obtained from each septic tank effluent for bacteriological analysis.

Sand Filter Operation--

Eight sand filters were established specifically for this study. Each filter had a surface area of 0.09 m^2, 30 cm of freeboard, 60 cm of sand underlain by 13 cm of pea gravel and 18 cm of coarse stone. All eight filters were housed in a single lysimeter constructed of plywood supported by an angle-iron frame, as shown in Figure A-73.

Portions of effluents from the 4.0 m^3 septic tanks were applied to the sand filters in several intermittent doses with a special distribution system. As shown in Figure A-73, the system was attached to the top of the sand filters and consisted of two identical chambers, one receiving grey water septic tank effluent and one receiving combined wastewater septic tank effluent. The operation of each chamber was essentially the same, and therefore, the following description will deal only with that for the grey water.

Each time the sump receiving the effluent from the grey water septic tank received about 28 L of effluent, a pump was activated with most of the wastewater discharged to the city sewer. However, a portion (adjustable) of the effluent was directed to the grey water distribution chamber through a 2.0 cm diameter section of plastic pipe connected to the sump discharge line. After three, 6 L fractions were so directed to the grey water chamber (four, 4.5 L portions for the combined wastewater) a high-level float switch was triggered, activating a relay which opened a 3.0 cm drain valve attached to the chamber bottom and also recorded the event. The effluent from the chamber flowed through the valve and into a fitting which divided the flow into 8, equal fractions. Each fraction was carried away from the fitting in a section of 0.6 cm diameter tygon tubing to drains or to an operating sand filter. In the latter case, the end of the tubing was held in place above the filter by insertion through a hole in a wood dowel attached to the top of the filter. The size of the dose and the daily hydraulic loading to a filter could be varied with this system by directing more than one section of tubing to it. The effluent flowing through each section of tubing was directed onto a splash plate on the sand surface. After this distribution chamber emptied,

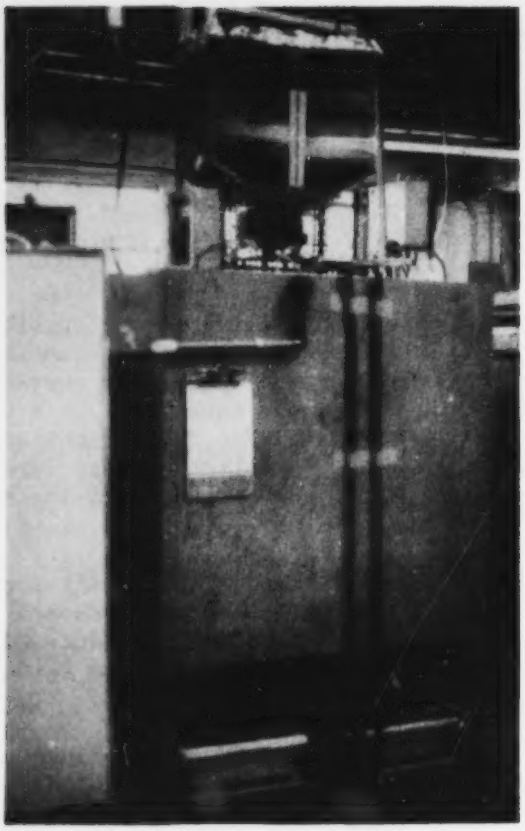


Figure A-73. Sand filter system.

a low-level float switch was triggered, deactivating the relay, closing the drain valve, and resetting the system.

The majority of the effluent applied to a filter drained through a 3.0 cm diameter line attached to its bottom, to a sump from which it was discharged to the city sewerage system. A fraction of the wastewater moving through a filter was collected near the filter bottom in four, reducer fittings (3.0 cm x 1.25 cm) whose upper lips were 5 cm above the filter bottom. The 1.25 cm end of each fitting projected through the filter bottom and a short section of 1.0 cm diameter tubing was attached to each. These four, sample collection tubes from each filter were directed to a collection tray for disposal or, as desired, to a sample bottle for a flow composited sample.

A summary of the experimental design used in the sand filter research is presented in Table A-149.

TABLE A-149. SAND FILTER EXPERIMENTAL DESIGN

Filter Number	Effluent Applied	Application Rate (cm/day)	Media Size	
			Effective Size (mm)	Uniformity Coefficient
1	Grey Water	30	.28	3.2
2	"	30	.30	2.9
3	"	15	.30	2.9
4	"	15	.28	3.2
5	Combined Wastewater	15	.28	3.2
6	"	15	.30	2.9
7	"	30	.30	2.9
8	"	30	.28	3.2

During the first two months of operation, the appropriate septic tank effluent was applied to each sand filter at the prescribed loading. In total, approximately 850 and 1700 L were applied, respectively. Then the loading was discontinued, the top 15 cm of sand were replaced, and the filters were rested for nine days prior to the start of actual data collection.

During the course of this study, daily sand filter loadings were recorded. Visual observations of the sand surfaces were routinely made and depth of any ponding above the sand surface was noted. Flow composited samples of the filter effluents were collected bi-monthly and analyzed for various chemical/physical parameters.

Results and Discussion

Septic Tank Performance--

The results of selected chemical/physical analyses on daily flow composited samples of each septic tank effluent are presented in Table A-150. The effluent from the 2.0 m^3 septic tank treating grey water was significantly higher in BOD_5 and COD than the 4.0 m^3 septic tank also treating grey water. For the other measured parameters, there was virtually no difference between the concentrations in the two, septic tank effluents. The increased residence time in the larger tank (about eight days versus four days) would seem to have been responsible for the greater BOD_5 and COD reductions, but of little additional value in terms of suspended solids removal or nutrient conversion. It is also possible that the physical characteristics of the smaller, circular tank could

TABLE A-150. SEPTIC TANK EFFLUENT COMPARISON, MG/L

Parameter	Grey Water 2 m^3	Grey Water 4 m^3	Combined Wastewater 4 m^3
BOD_5[*]	101(44)[**] 88-114	62(57)† 56-68	55(60)† 50-61
COD	236(42)† 220-252	171(55)† 159-183	169(58)† 155-185
TSS	47(41)† 42-53	46(56)† 41-51	46(58)† 41-52
VSS	37(40)† 32-41	34(55)† 32-37	33(58)† 30-37
Total-N	6.5(22) 6.1-7.0	7.7(22) 7.0-8.4	79(22) 75-84
NH_3-N	1.4(25) 0.9-1.8	2.1(39) 1.7-2.5	54(43) 49-59
Total-P	44(28) 41-47	40(30) 37-42	43(27) 40-45
Ortho-P	34(27) 29-38	34(26) 31-37	36(27) 33-39

[*] Based on 10 sample analyses, soluble BOD_5 to total BOD_5 equal to 0.8 for all units.

[**] Each data block includes: Mean (samples); 95% confidence interval.

† Log-normalized data.

have resulted in the tank effluent being more influenced by surge loadings than the larger cylindrical tank.

Although the concentrations of BOD_5, COD and suspended solids were considerably higher in the raw combined wastewater as compared to the raw grey water (Table A-150) the measured characteristics of the effluents from the two, 4.0 m^3 septic tanks were essentially the same. The grey water tank provided an average retention time of eight days while the combined wastewater tank provided five days. Visual inspection of the two septic tank effluents during the course of the study revealed that the grey water tank consistently produced an effluent which was grey-black in overall color with substantial quantities of fine, black solids. The combined wastewater tank produced an effluent which had a lighter color with significantly less fine, black solids. The black coloration and fine, black solids was believed to be due to the reduction of sulfates (approximate tap water concentration, 80 mg/L as $SO_4^=$) to sulfides under anaerobic conditions, with resultant precipitation as metal sulfides. A sulfide odor was noted in both septic tank effluents, but the odor was considerably stronger in the effluent from the grey water tank. The pH of both effluents was about 7.3.

The results of bacteriological analyses on several grab samples of each septic tank effluent are presented in Table A-151. The results for the septic tanks receiving grey water were not anticipated. After April, 1976, no material of fecal origin was added to either grey water tank. It was anticipated that initially, indicator bacteria would be present in the septic tank effluents as a result of pre-sampling operations during which both tanks received combined wastewaters. At the start of this study, the grey water tank was not pumped or cleaned, while the smaller grey water tank was washed down and cleaned out, except for a few cm of residual material. With time and the dilution effected by the influent grey waters, it was expected that the concentration of indicator bacteria in the septic tank effluents would decrease to low levels. As shown in the table, this did not happen. Even after seven and thirteen months of operation, very high levels of indicator bacteria were found in the grey water tank effluents. It would seem that the indicator bacteria were actively reproducing in these tanks. If the traditional indicator bacteria do, in fact, actively reproduce in a grey water septic tank environment, one must question the value of using the effluent concentrations of these organisms to assess the potential for pathogenic contamination of the raw grey waters.

Sand Filter Performance--

Each of the eight sand filters was operated until its hydraulic conductivity decreased to the point where it no longer accepted the wastewater applied and ponding above the sand surface reached a height of about 25 cm. When this occurred, loading to the filter was discontinued and it was allowed to drain. Since only the four high-rate filters (1, 2, 7, 8) failed during the course of this study, the following discussion will be restricted to these four filters. A summary of the operational data for each filter and its filter run is presented in Table A-152.

As presented in the table, the filters receiving grey water septic tank effluent yielded filter run lengths over twice those of the filters receiving combined wastewater septic tank effluent. As the hydraulic loading rates were approximately equal, the grey water filters (1, 2) accepted about twice as much wastewater on a total volume basis. Utilizing the mean characteristics of each applied wastewater (grey water and combined wastewater septic tank effluents), the quantities of BOD_5 and suspended solids per square metre of surface area were calculated. The grey water filters received about 140 percent more BOD_5 and 60 percent more suspended solids than did the combined wastewater filters (Table A-152). Since the effluents from the four filters were of very similar quality, the grey water filters effectively removed approximately 140% more BOD_5 and 60% more suspended solids than the combined wastewater filters.

TABLE A-151. BACTERIOLOGICAL CHARACTERISTICS OF SEPTIC TANK EFFLUENTS, NO./100 ML

Organism	Sample Date	Grey Water 2 m^3	Grey Water 4 m^3	Combined Wastewater 4 m^3
Total Coliform	7-76	-	33×10^4	49×10^5
	11-76	15×10^4	12×10^4	12×10^4
	5-77	-	54×10^5	50×10^5
Fecal Coliform	7-76	-	33×10^4	49×10^5
	11-76	34×10^3	49×10^3	31×10^3
	5-77	-	36×10^5	14×10^5
Fecal Streptococci	7-76	-	-	-
	11-76	<100	18×10^3	48×10^2
	5-77	-	23×10^2	34×10^4
Total Bacteria	7-76	-	31×10^7	11×10^8
	11-76	40×10^6	41×10^5	16×10^6
	5-77	-	33×10^7	16×10^9

TABLE A-152. HIGH-RATE FILTER OPERATION

Filter	Effluent Applied	Daily Application		Run Days to-		Total Application / m^2		
		Doses / Day	cm/day	Ponding	Failure	L	Grams	Grams
1	Grey	5	29	226	264	6610	4410	3230
2	Grey	5	29	260	285	7110	4740	3550
7	Combined	6	31	104	124	3330	1940	2040
8	Combined	6	31	108	124	3310	1940	2040

During the operation of the filters prior to failure, a distinct visual difference was noted between the surfaces of the grey water and combined wastewater filters. The surfaces of both high-rate grey water filters exhibited significantly less solids accumulation and surface impeding layers than did the high-rate combined wastewater filters. After a filter failed and was allowed to drain, visual inspection of the sand surface was made. Subsequently, the filter was carefully excavated from the surface down to allow observation of any subsurface impeding layers and the filter profile with depth.

The combined wastewater filters had a thin mat of solids covering a majority of the filter surface. Upon digging into each filter, a black colored zone was found to extend to a depth of several cm followed by a gradation through grey to visually clean sand. In contrast, the grey water filters had only a sparse accumulation of solids on their surfaces. Upon excavation, a black colored zone was found to exist to a depth of about 25 cm before gradation to visually clean sand.

The black colored zones within these filters are most likely due to reduction of sulfates under anaerobic conditions during ponding with the resultant precipitation and accumulation of metal sulfides on the sand particles. The deeper penetration of the black zones in the grey water filters may have been due to the more reduced state of the wastewater initially, as well as the greater ponding times (38 to 25 days for 1 and 2 versus 20 and 16 days for 7 and 8).

The results of analyses for selected chemical/physical parameters on daily flow composited samples of the effluents from each of the eight sand filters is presented in Table A-153. As shown, filtration through 60 cm of medium sand at an application rate of 15 to 30 cm/day provides a high quality effluent with regard to BOD_5 and volatile suspended solids, with values consistently less than 5 mg/L and 10 mg/L, respectively.

TABLE A-153. SAND FILTER EFFLUENT CHARACTERISTICS

Parameter	Grey Water				Combined Waste			
Loading cm/day	1	2	3	4	5	6	7	8
	30	30	15	15	15	15	30	30
BOD_5*	1(17)** 1-3	1(18) 1-3	1(17) 1-3	1(18) 1-2	2(19) 1-3	1(13) 1-3	4(15) 2-7	4(20) 2-6
COD	26(6) 13-39	17(7) 7-27	21(7) 7-35	16(7) 12-20	16(9) 10-23	18(9) 10-25	25(6) 9-40	57(9) 29-86
SS*	12(19) 9-16	14(19) 10-19	11(16) 7-16	8(20) 6-10	8(20) 5-12	15(13) 9-23	18(14) 11-31	17(20) 12-25
VSS	8(19) 6-9	7(19) 5-9	7(16) 5-9	5(20) 4-6	5(20) 3-6	6(13) 4-9	10(15) 6-14	8(20) 5-10

* Log-normalized data

** Each data block includes: Mean (samples); 95% confidence interval

Further, the nitrogen in the septic tank effluents, largely as ammonia, was almost completely nitrified, while there was no significant effect on the phosphorus concentrations as a result of the sand filtration.

Summary

The results of this study may be summarized as follows:

1. The reduction in BOD_5 and COD are considerably greater in grey water when treated with a 4.0 m^3 septic tank as compared to a 2.0 m^3 septic tank. However, with regard to other chemical/physical parameters such as suspended solids, nitrogen and phosphorus, there is no appreciable difference.

2. Bacteria used as indicators of fecal contamination were seen to maintain high concentrations in the effluents from septic tanks receiving just grey water after as much as thirteen months of operation without a fecal input. Residual material in the septic tanks from a previous study during which they received combined wastewater, would seem to have provided the seed from which the bacteria reproduced to maintain the high levels.

3. Sand filters receiving grey water septic tank effluent yielded filter run lengths over twice as long, and removed over 140% more BOD_5 and 60% more suspended solids than did similar filters receiving combined wastewater septic tank effluent (dosed 4 to 5 times daily with a total loading of about 30 cm/day).

4. A distinct difference was noted, both on the surface and with depth, between sand filters loaded with grey water septic tank effluent as compared to combined wastewater septic tank effluent.

5. Intermittent sand filtration of both grey water septic tank effluent through 60 cm of medium sand at application rates of about 15 and 30 cm per day, produced effluents low in BOD_5 and suspended solids, almost completely nitrified, with the phosphorus largely unchanged.

As indicated by the results of this preliminary laboratory study, further research is necesssary regarding grey water treatment and disposal. Research which would 1) further identify the characteristics of grey water septic tank effluents, 2) identify the efficiency of sand filtration of grey water with regard to chemical/physical and microbiological parameters, 3) delineate the most appropriate filter operating conditions and maintenance techniques, and 4) investigate the development of clogging and occurrence of failure in sands and other soils receiving grey water septic tank effluent as compared to combined wastewater septic tank effluent.

PROCESS COSTS

The evaluation of on-site waste treatment and disposal alternatives includes an analysis of technological effectiveness, institutional arrangements and owner acceptance and cost effectiveness. The first of these general areas of evaluation have been discussed at length in the preceding sections; the second will be discussed in detail in Appendix D. Information on cost evaluation of potential on-site treatment and disposal alternatives is meager at the present time owing to the small data base. The information presented herein is based, primarily upon the experiences of this research effort. Costs will be subdivided into capital investment plus installation, operation and maintenance costs. Range of costs will be presented in most instances to reflect a variety in labor, power and material costs. Costs will be based on 1977 dollars.

In-House Alteration Devices

The research studies reported herein did not include an in-depth cost analysis of the extensive array of devices and systems which provide for waste flow reduction and/or waste segregation are commercially available. Cost information regarding their processes may be found elsewhere (Wallman and Cohen, 1974; Milne, 1976; Nelson, 1976; Reid, 1976).

In general, the costs for these types of in-house devices vary dramatically from virtually zero capital and operating costs for the simple flow control faucet inserts, to as much as $5000 in capital and $125 per year in operating costs for the sophisticated closed-loop recycle toilets. In identifying costs for these types of in-house devices, caution must be exercised as the unit costs for a given type of device can vary significantly depending on many factors, including 1) the commercial manufacturer, 2) the location of the nearest distributor, 3) the method of installation (self-installed vs. hired labor), and 4) the costs for utility service (e.g., power and water).

Treatment Processes

Septic Tanks--

Costs of septic tanks vary with the materials used and the site conditions. The delivered cost of precast concrete septic tanks in Wisconsin (1977) can be calculated by the approximate formula,

$$C = \$200 + 0.20(V-750)$$

in which C is the delivered cost and V is the capacity in gallons (x 3.75 liters). This formula is good up to 6975 L (1800 gal) tanks. Above that, cast-in-place tanks are used. Similar costs are found for steel tanks.

Installation costs including building sewer from the home are approximately $100-$200. Although no maintenance costs were incurred in this study, it may be assumed that pumping would be required every 3 years. In Wisconsin, pumping costs for a 3.75 m^3 (1000 gal) tank would range from $40-$60. Based on these figures, the range of unit costs for a 3.75 m^3 (1000 gal) septic tank would be as follows:

3.75 m^3 tank, installed	$350 - $450
Operation	0
Maintenance	$12-$20/yr.

Aerobic Units--

The cost for aerobic units is high due to a generally low demand for these units. Costs could reduce with increased market penetration. Presently, costs range from approximately \$1200 to \$2200 for a home-sized package aeration unit, depending upon propietary device and location. Installation costs are normally higher than for septic tanks owing to the greater complexity of the unit and the necessary electrical work. These costs may range from \$200-\$300. Power consumption for these units also vary, ranging from 2.4 to 7.4 kwh/day. Other costs include costs for pumping, which should be performed at least yearly and preventative maintenance services performed approximately 4 to 6 times per year. For pumping costs, a range of from \$40-\$60 is assumed and additional maintenance costs may range from \$25-\$50/yr.

Based on these figures, the range of unit costs expected for a package aerobic unit for the average home would be as follows:

Package aerobic unit, installed	\$1500-\$2500
Operation (Power: 2.4 to 7.4 kwh/d @ 4¢/kwh)	\$35-\$108
Maintenance (1 pumping per year plus service calls	\$65-\$110

Chemical Precipitation/Coagulation--

Little information is available on actual capital costs for household chemical treatment units. A hydraulically driven unit could be purchased for approximately \$700-\$800 which includes a settling chamber, but not the treatment unit preceding it. Operating costs would be dependent upon chemical costs. For alum, approximately 200 mg/L would be used which would amount to about 0.15 kg/day for a household of 4. Iron salts for coagulation and phosphorus removal would require similar dosage. Detailed chemical cost analysis is beyond the scope of this report but crude estimates based on the figures above and the costs given in Table A-154 would indicate a range of from \$12 to \$60/yr. Sludge pumping would likely be required at least once per year. Maintenance calls would likely be required at least 4 times/yr. Based on these crude estimates, chemical coagulation unit costs would be estimated as follows:

Mechanical unit, installed	\$800-\$1000
Operation Costs (Chemicals)	\$12-\$60
Maintenance (includes 1 pumping/yr plus 4 maintenance calls)	\$65-\$110

Sand Filters--

Based on Wisconsin experience, construction costs including a precast concrete filter shell (septic tank), plumbing, media, insulated cover and installation could range from \$15 to \$20/ft^2 of filter area. Additional costs for dosing sump and pump are not included (see below). Maintenance costs

TABLE A-154. CHEMICAL COSTS (1977)

Chemical	Cost (FOB)-$
Alum (dry, bulk)	6.60-7.70/100 kg
$FeCl_3$ (liquid, bulk as $FeCl_3$)	8.80-11.00/kg
Lime (dry pebble, bulk as CaO)	2.20-2.75/100 kg
$FeSO_4 \cdot 7\ H_2O$ (dry, bulk, 21% Fe#)	2.20-2.65/100 kg
Calcium hypochlorite (70% HTH)	170/100 kg
Calcium hypochlorite tablets (70% HTH)	420/100 kg
Iodine (elemental)	600/100 kg
Iodophor (1.75%)	1.65/liter
Gluteraldehyde (50% solu)	121/100 kg
Formaldehyde (37% solu)	35/100 kg
KOH (45% dry, bulk)	16.50/100 kg
NaOH (50% dry, bulk)	15.40/100 kg

would vary depending upon pretreatment unit and techniques but can be estimated at \$.75 to \$1.25/ft^2/yr.

Based on these figures, the unit cost range for household intermittent sand filters will be as follows:

Equipment and installation - \$15-\$20/ft^2

Maintenance costs - \$0.75-\$1.25/yr

Pump Chamber and Sump Pump--

Costs for pumping chambers required to precede sand filters, or other unit processes are based on Wisconsin experience. Installed cost for a 1.5 m (5 ft) section of 1.2 m (48 in) diameter concrete pipe with poured bottom slab and plumbing appurtenances would be approximately \$200-\$500. A 0.37 kw (1/2 hp) submersible pump with controls is estimated at \$300-\$500. Power demand is estimated at 0.1 kwhr/day.

Based on these figures, the cost for a dosing chamber would be approximated as follows:

Pump Chamber, installed	$200-$250
Sump Pump with controls	$300-$350
Operation - (0.1 kwh/d @ $0.04/kwh)	$1.50/yr.
Maintenance Costs (1 hour/yr)	$6-$10/yr.

Chlorination and Contact Chamber--

Chlorination of wastewater prior to surface discharge may be accomplished through the use of simple tablet feed devices. Based upon field studies at several locations in Wisconsin, it is estimated that approximately .015 kg/day of tablet should be used at a cost of $4.20/kg for a household of 4. In addition to the basic tablet feed chlorination system, which may cost from $150-$175 a contact tank (1.2 m dia. concrete pipe at 2.75 m length plus slab and appurtenances at $300) is required. Sometimes a 1/3 hp sump pump to discharge the wastewater from the contact tank to the surface recipient ($100-$150) may also be required.

The estimated cost for a chlorination system is as follows:

Chlorine contact tank plus installation including plumbing and excavation	$400-$500
1/3 hp pump and controls	$100-$150
Chlorinator - tablet feed	$150-$175
Maintenance Costs (1 hour/yr)	$6-$10/yr.
Operation Costs	
Electrical power - (0.1 kwh/d x $0.04)	$1.50
Hypochlorite tablets (0.015 kg/d @ $4.20/kg)	$26/yr.

Ultraviolet Disinfection--

The ultraviolet system basically consists of a staging tank for pumping effluent to the unit, a sump pump and a UV sterilizer. The staging tank can be a 0.9 m dia. concrete pipe 2.75 m in length with poured slab. There are a number of propietary UV disinfection units on the market today with costs ranging from $750-$900. Little experience has yet been gained with maintenance costs but unit cleaning is paramount to effective disinfection of wastewater. System costs estimated below are based on disinfection of sand filtered effluent for a flow of approximately 750 L/day.

Staging tank plus installation	$300-$450
1/3 hp submersible pump and control	$100-$150
UV sterilizer unit	$750-$900
Maintenance (10 hours per year)	$40-$60/yr.
Operation -	
Power (1.5 kwh/d @ 4¢/kwhr)	$20/yr.

Septage Disinfection--

Based upon laboratorty experiments with septage reported in this study (Deininger, 1977) an estimate of cost of septage disinfection was forecasted. These costs are based upon chemical costs given in Table A- . This estimate does not include ultimate disposal costs which are estimated from $40-$60 every three years. Costs for three disinfection alternatives for 3750 L (1000 gal) of septage are shown in Table A-154.

TABLE A-155. SEPTAGE DISINFECTION COSTS

Disinfectant	Dose Level mg/L	Cost to Treat 3750 L (1000 gal) of Septage
Glutaraldehyde	500	$4.58
(no pH adjustment)	1,000	$9.17
Formaldehyde	500	$4.54*
(sludge adjusted to pH 10)	1,000	$6.28*
Chlorine (no pH adjustment)	2,000 **	$18.72

* Including pH adjustment estimate with KOH (Cost = $2.79)

** The highest chlorine dose tested; provided marginal treatment of 40,000 mg/L TS sludge.

Soil Disposal Costs--

Soil absorption fields--Based on Wisconsin experience, soil absorption field construction costs currently range from $1 to $1.25/ft^2 of bottom area depending upon soil type and location. No maintenance costs have been identified for these systems.

Mounds - There is now a substantial body of data available on cost of mound construction in Wisconsin. Installed costs including septic tank, sump and pump, mound construction, plumbing and landscaping range from $3000 to $4500. Maintenance costs include those associated with septic tank pumping and occasional maintenance of the sump pump. Estimates for maintenance are in the range of $18-$25/year. Power consumption costs for pumping to the mound are based on approximately 0.25 kwh/day consumption.

Estimated mound unit costs range would be as follows:

Installation and equipment costs	\$3000-\$4500
Maintenance	\$18-\$25/yr.
Power - 0.1 kwh/d @ 0.04/kwh	\$1.50/yr.

ATTACHMENTS TO APPENDIX A

Attachment A - Individual Home Water Use Patterns

Attachment B - Statistical Analysis Results for Each Event, mg/cap/day

Attachment C - Literature Reviewed and Contacts Made

Attachment D - Description of Laboratory Site N and the Influent Wastewater

ATTACHMENT A - INDIVIDUAL HOME WATER USE PATTERNS

Daily Flow Patterns

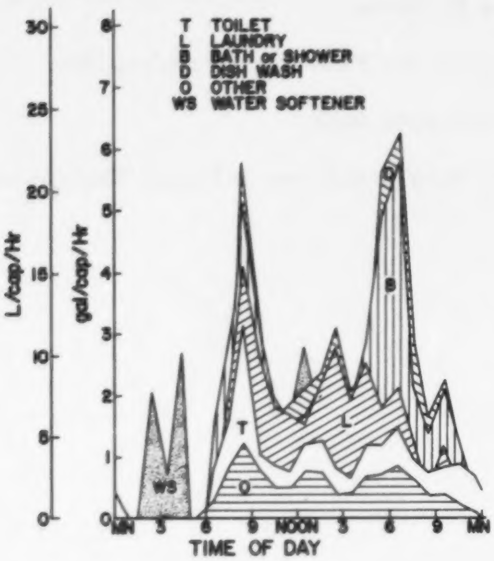


Figure A-74. Family A daily flow pattern (L/cap/day = 214).

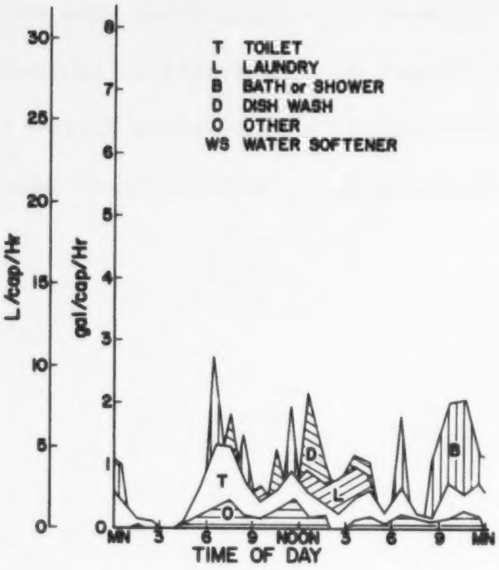


Figure A-75. Family B daily flow pattern (L/cap/day = 96).

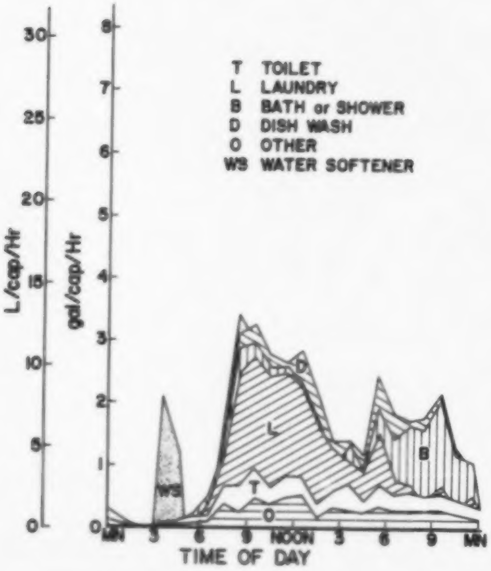


Figure A-76. Family C daily flow pattern (L/cap/day = 147).

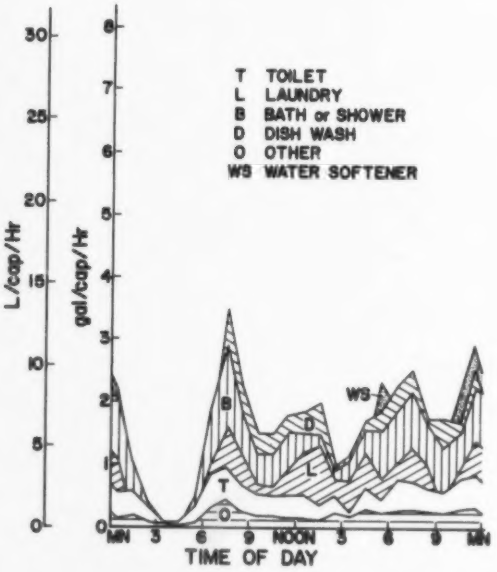


Figure A-77. Family D daily flow pattern (L/cap/day = 155).

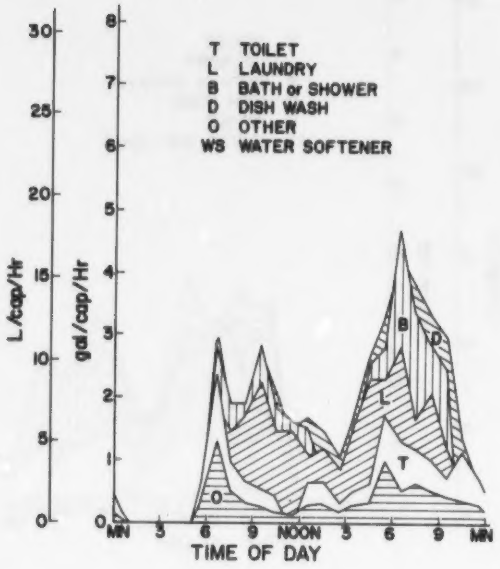


Figure A-78. Family E daily flow pattern (L/cap/day = 157).

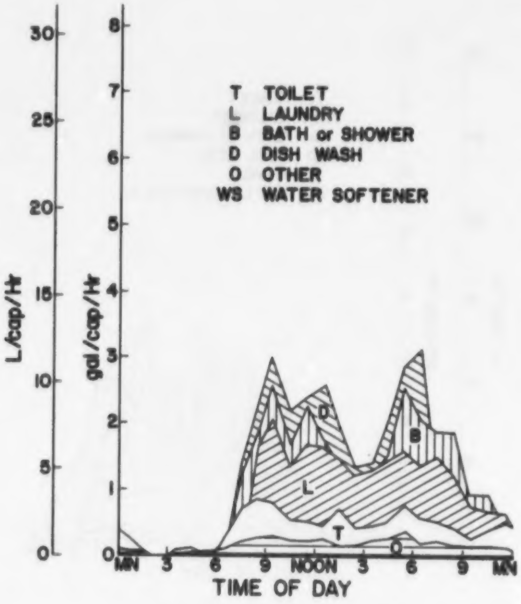


Figure A-79. Family F daily flow pattern (L/cap/day = 125).

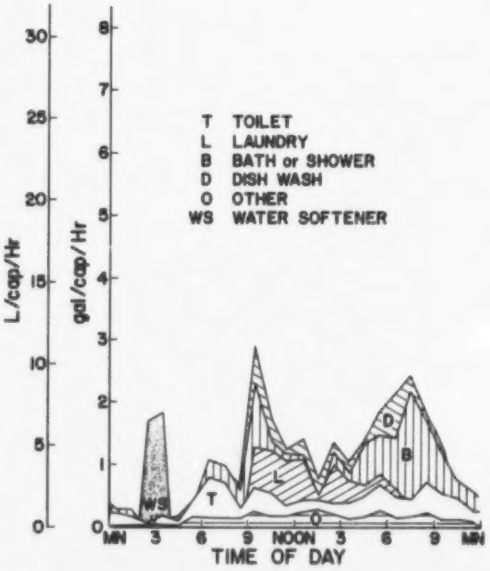


Figure A-80. Family G daily flow pattern (L/cap/day = 111).

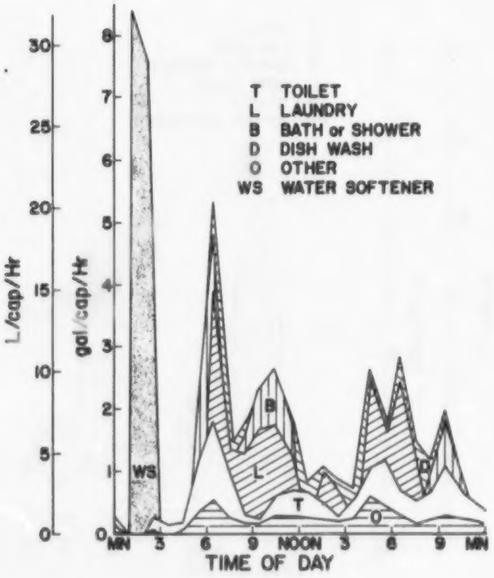


Figure A-81. Family H daily flow pattern (L/cap/day = 188).

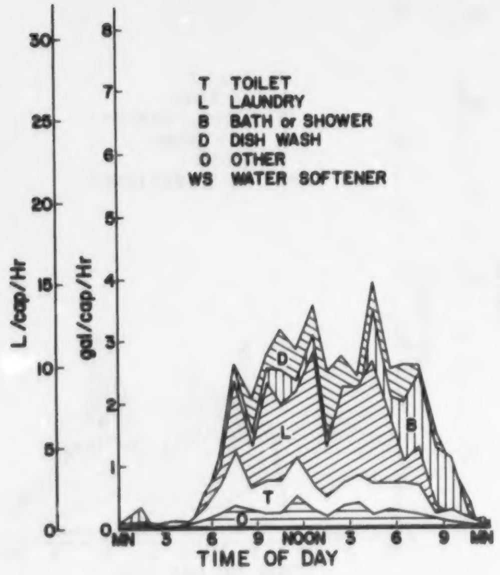


Figure A-82. Family I daily flow pattern (L/cap/day = 158).

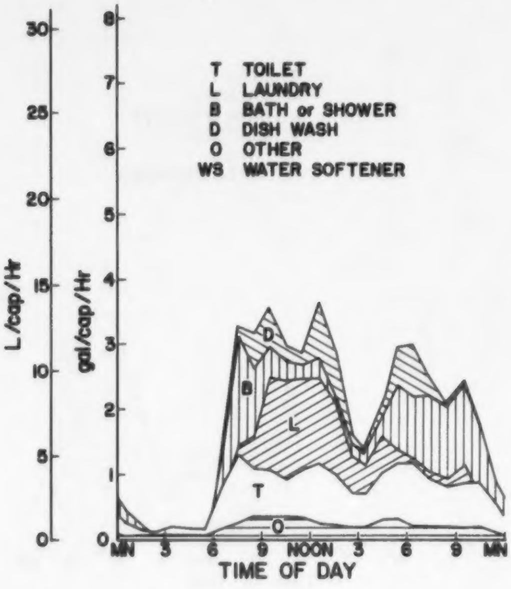


Figure A-83. Family J daily flow pattern (L/cap/day = 170).

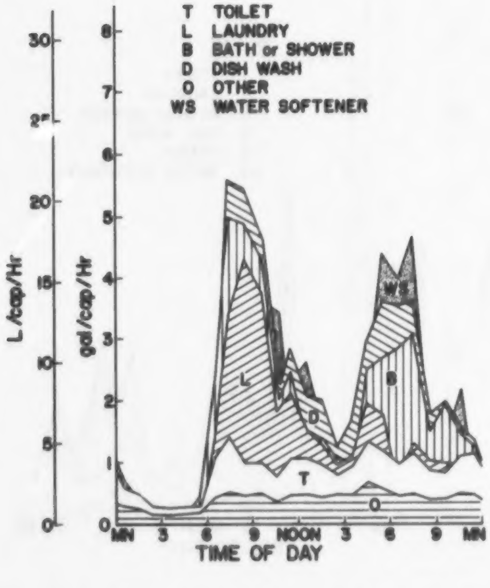


Figure A-84. Family K daily flow pattern (L/cap/day = 215).

Weekly Flow Patterns

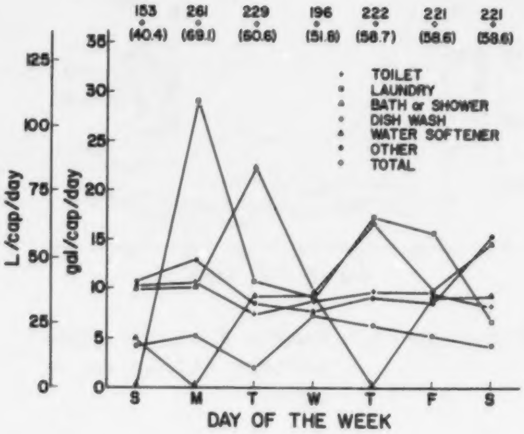


Figure A-85. Family A weekly flow pattern (L/cap/day = 214).

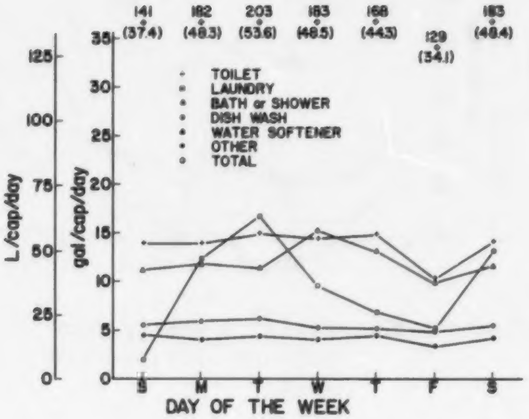


Figure A-86. Family B weekly flow pattern (L/cap/day = 96).

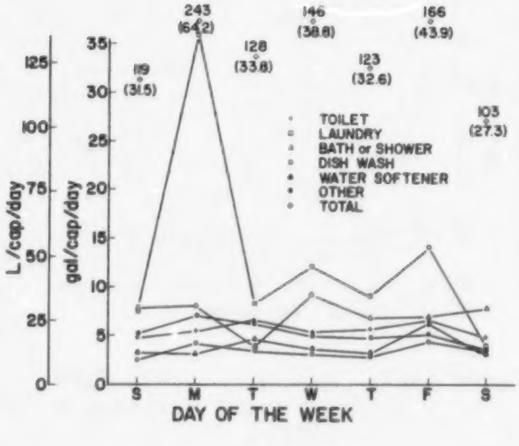


Figure A-87. Family C weekly flow pattern (L/cap/day = 147).

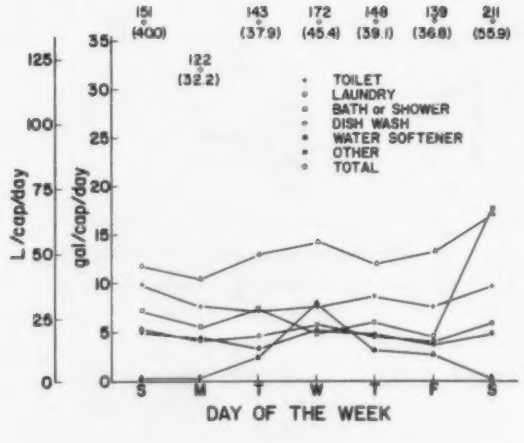


Figure A-88. Family D weekly flow pattern (L/cap/day = 155).

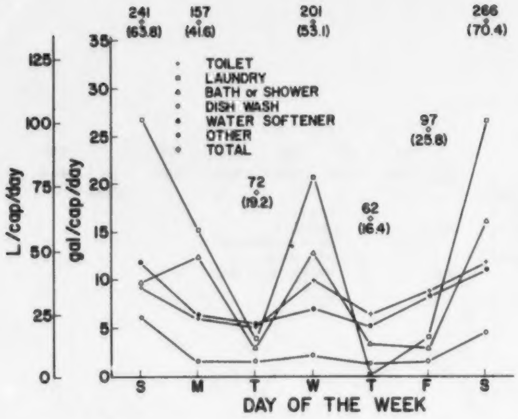


Figure A-89. Family E weekly flow pattern (L/cap/day = 157).

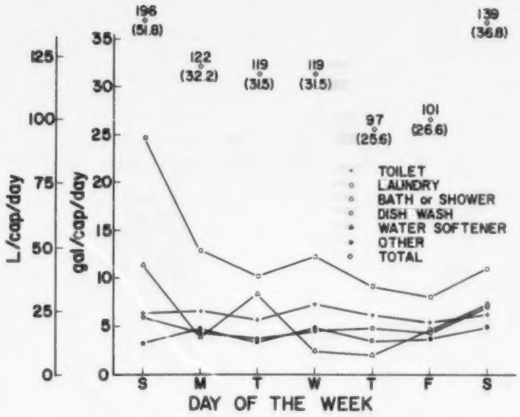


Figure A-90. Family F weekly flow pattern (L/cap/day = 125).

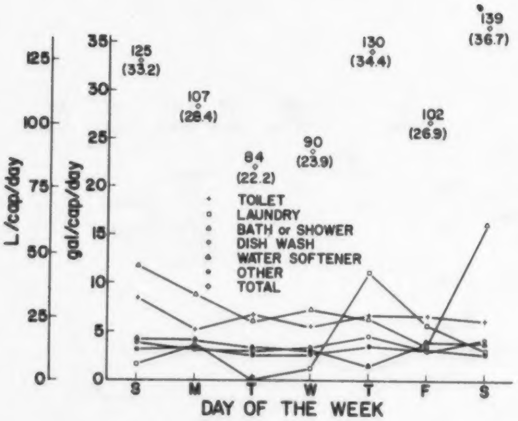


Figure A-91. Family G weekly flow pattern (L/cap/day = 111).

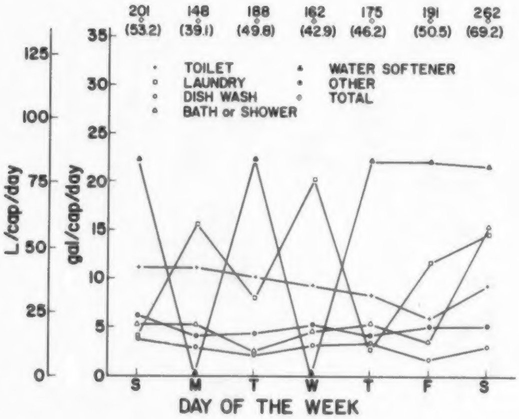


Figure A-92. Family H weekly flow pattern (L/cap/day = 188).

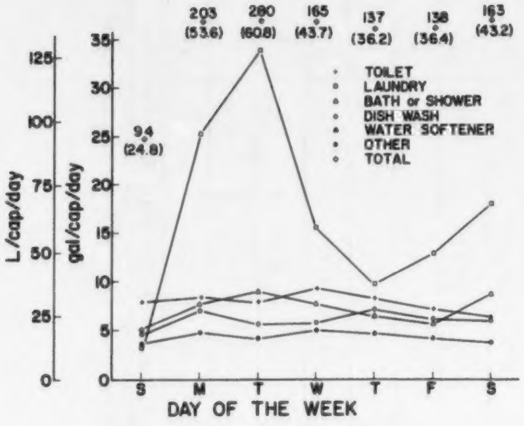


Figure A-93. Family I weekly flow pattern (L/cap/day = 158).

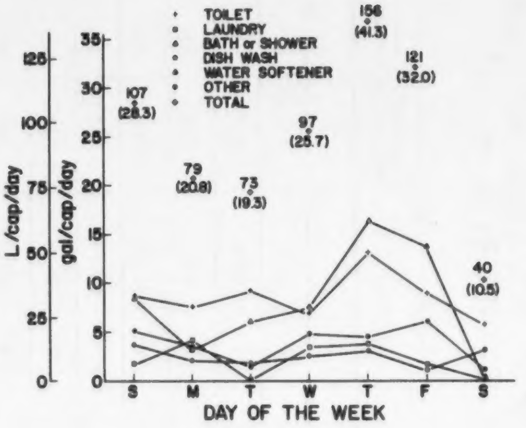


Figure A-94. Family J weekly flow pattern (L/cap/day = 170).

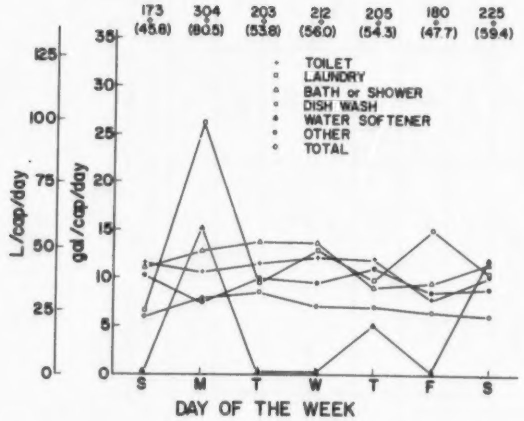


Figure A-95. Family K weekly flow pattern (L/cap/day = 215).

TABLE A-156. FECAL TOILET FLUSH WASTEWATER CHARACTERISTICS, mg/cap/day*

Parameter	Mean	Range	Standard Deviation	Data Points
Flow	7.1	6.1 — 9.5	1.0	42
BOD_5 U	4337	857 — 10167	2250	35
BOD_5 F	2339	516 — 5755	1383	32
TOC U	3533	648 — 7172	1797	32
TOC F	1574	324 — 4165	880	32
TS	1059	4004 — 21496	4736	35
TVS	7757	2446 — 17931	3865	35
TSS	6240	993 — 15244	3948	33
TVSS	5087	889 — 13800	3334	34
TOT-N	1497	240 — 3326	855	35
NH_3-N	593	53 — 1593	486	32
NO_3-N	6.3	0 — 22	6.0	41
TOT-P	268	55 — 899	185	40
ORTHO-P	115	29 — 455	83	38
Grease	928	428 — 1488	382	8
Temp.	66	56 — 73	4.8	19

* All values are in mg/capita/day except flow (liters/capita/day) and temperature (°F). All values have had carriage water contributions removed except grease and temperature.

TABLE A-157. NONFECAL TOILET FLUSH WASTEWATER CHARACTERISTICS, mg/cap/day*

Parameter	Mean	Range	Standard Deviation	Data Points
Flow	19.5	12.3 — 25.7	3.2	41
BOD_5 U	6379	948 — 15203	3929	35
BOD_5 F	3979	631 — 10840	2493	32
TOC U	4246	751 — 9693	2467	31
TOC F	3165	320 — 6809	2046	24
TS	17799	4308 — 40046	9945	35
TVS	11972	3171 — 29876	7867	33
SS	6276	2488 — 15610	3897	34
VSS	5122	327 — 12934	3666	33
TOT-N	2644	326 — 8777	2078	37
NH_3-N	521	32 — 2067	619	36
NO_3-N	21.1	0 — 92	22	37
TOT-P	280	98 — 1282	231	35
ORTHO-P	188	22 — 548	123	36
Grease	2423	1270 — 3745	936	7
Temp.	66	56 — 73	4.8	19

* All values are in mg/capita/day except flow (liters/capita/day) and temperature (°F). All values have had carriage water contributions removed except grease and temperature.

TABLE A-158. GARBAGE DISPOSAL WASTEWATER CHARACTERISTICS, mg/cap/day*

Parameter	Mean	Range	Standard Deviation	Data Points
Flow	10.6	7.1 — 14.3	2.5	8
BOD_5 U	10923	1631 — 34688	13462	7
BOD_5 U	2568	1136 — 6544	2116	6
TOC U	7317	1817 — 16781	5063	7
TOC F	3911	823 — 10819	3678	6
TS	25755	7031 — 43875	15289	6
TVS	24018	6675 — 42562	14096	6
SS	15823	5325 — 28125	10439	5
VSS	13486	2076 — 24600	10087	5
TOT-N	632	103 — 1549	513	6
NH_3-N	9.6	.8 — 34	13.9	5
NO_3-N	.20	0 — .8	.4	4
TOT-P	128	26 — 339	104	7
ORTHO-P	88	13.1 — 227	80	6
Grease	2100	1830 — 2370	382	2
Temp.	71	64 — 80	5.5	7

*All values are in mg/capita/day except flow (liters/capita/day) and temperature (°F). All values have had carriage water contributions removed except grease and temperature.

TABLE A-159. KITCHEN SINK WASTEWATER CHARACTERISTICS, mg/cap/day*

Parameter	Mean	Range	Standard Deviation	Data Points
Flow	5.7	3.3 — 10	2	12
BOD_5 U	8344	3158 — 16514	3708	10
BOD_5 F	4576	1999 — 7202	1650	7
TOC U	5000	1513 — 7869	1791	11
TOC F	4111	1135 — 7789	1665	11
TS	13761	5653 — 22826	5638	11
TVS	9731	2797 — 15348	4041	11
SS	4111	1347 — 7287	2425	11
VSS	3841	974 — 7095	2400	11
TOT-N	424	157 — 772	224	10
NH_3-N	32.3	12.8 — 65	20.4	9
NO_3-N	1.8	0 — 8.3	2.5	10
TOT-P	419	21.8 — 1587	519	10
ORTHO-P	177	5.2 — 482	180	10
Grease	2329	468 — 5771	1753	8
Temp.	80	72 — 94	6.9	10

*All values are in mg/capita/day except flow (liters/capita/day) and temperature (°F). All values have had carriage water contributions removed except grease and temperature.

TABLE A-160. AUTOMATIC DISHWASHER WASTEWATER CHARACTERISTICS, mg/cap/day*

Parameter	Mean	Range	Standard Deviation	Data Points
Flow	12.1	7.0 — 16.9	3.9	15
BOD_5 U	12625	2841 — 24893	7253	14
BOD_5 F	7835	2113 — 21534	5243	14
TOC U	7276	2162 — 17443	4318	13
TOC F	4686	1136 — 14686	3904	14
TS	18157	7413 — 29975	6791	13
TVS	10544	2552 - 21965	5099	13
SS	5267	1030 — 9527	2718	14
VSS	4457	564 - 8717	2630	14
TOT-N	487	127 - 1163	326	14
NH_3-N	54	2.6 — 172	57	13
NO_3-N	4.1	0 — 12.1	3.2	15
TOT-P	819	110 - 1405	387	15
ORTHO-P	382	89 - 775	216	15
Grease	2476	1113 — 3296	1189	3
Temp.	101	70 — 125	23	12

* All values are in mg/capita/day except flow (liters/capita/day) and temperature (°F). All values have had carriage water contributions removed except grease and temperature.

TABLE A-161. AUTOMATIC CLOTHESWASHER - RINSE CYCLE WASTEWATER, mg/cap/day*

Parameter	Mean	Range	Standard Deviation	Data Points
Flow	28	6.8 — 40.1	10.7	27
BOD_5 U	10763	1552 — 26863	7071	24
BOD_5 F	6965	742 - 21267	5064	25
TOC U	7698	849 — 17246	4837	25
TOC F	5381	849 — 11927	2884	24
TS	37489	3443 - 94501	26178	27
TVS	14657	1630 — 32780	8475	25
SS	7927	752 — 17908	5030	26
VSS	4699	558 — 14759	3785	27
TOT-N	579	130 — 1423	362	24
NH_3-N	19.4	0 — 80	19.4	24
NO_3-N	17.	0 - 48	16.8	26
TOT-P	1602	167 - 4675	1233	25
ORTHO-P	411	59 — 1119	259	25
Grease	1844	570 — 2955	1035	4
Temp.	90	74 - 115	8	17

* All values are in mg/capita/day except flow (liters/capita/day) and temperature (°F). All values have had carriage water contributions removed except grease and temperature.

TABLE A-162. AUTOMATIC CLOTHESWASHER WASH-CYCLE WASTEWATER, mg/cap/day*

Parameter	Mean	Range	Standard Deviation	Data Points
Flow	26.5	6.3 — 42.4	11.0	29
BOD_5 U	4011	664 — 9877	3153	26
BOD_5 F	2842	489 — 7533	2230	26
TOC U	2605	146 — 8235	2293	25
TOC F	1907	137 — 7675	1860	24
TS	10941	1952 — 33339	8484	25
TVS	4794	1695 — 14567	3194	26
TSS	3043	317 — 8043	2032	28
TVSS	1817	252 — 5117	1318	28
TOT-N	146	34 — 297	94	27
NH_3-N	11.4	.1 — 37	8.4	27
NO_3-N	10.3	0 — 33	11.0	25
TOT-P	548	122 — 1855	489	26
ORTHO-P	112	11.7 — 416	102	26
Grease	1406	192 — 3006	1282	4
Temp.	83	72 — 95	6	15

* All values are in mg/capita/day except flow (liters/capita/day) and temperature (°F). All values have hac carriage water contributions removed except grease and temperature.

TABLE A-163. BATH/SHOWER WASTEWATER CHARACTERISTICS, mg/cap/day*

Parameter	Mean	Range	Standard Deviation	Data Points
Flow	18.5	4.7 — 31.7	8.3	27
BOD_5 U	3086	794 — 6939	2144	22
BOD_5 F	1872	351 — 4991	1451	20
TOC U	1749	185 — 4005	1179	20
TOC F	1128	100 — 2369	719	18
TS	4590	1115 — 14697	3726	22
TVS	3596	849 — 11704	2997	21
TSS	2261	608 — 7095	1824	20
TVSS	1571	502 — 4764	940	22
TOT-N	306	68 — 970	240	24
NH_3-N	40	1.7 — 136	37	22
NO_3-N	7.4	0 — 31	8.2	24
TOT-P	36	2.3 — 125	35	19
ORTHO-P	21	1 — 92	22.7	19
Grease	3219	1224 — 5363	1801	4
Temp.	85	72 — 90	4.7	18

* All values are in mg/capita/day except flow (liters/capita/day) and temperature (°F). All values have hac carriage water contributions removed except grease and temperature.

ATTACHMENT C - LITERATURE REVIEWED AND CONTACTS MADE

TABLE A-164. JOURNALS, ABSTRACTS AND INDEXES REVIEWED

Literature	Date Covered
Pollution Abstracts	1970 - December, 1976
Selected Water Resources Abstracts	1968 - December, 1976
Applied Science and Technology Index	1965 - December, 1976
Chemical Abstracts	1962 - December, 1976
NTIS Reports	1964 - April, 1975
EPA Reports Quarterly	1972 - September, 1976
Journal, Water Pollution Control Federation	1928 - October, 1976
ASCE Environmental Engineering Division Journal	1965 - December, 1976
Public Works	1965 - June, 1976
Water and Sewage Works	1967 - June, 1976
American Water Works Association	1971 - June, 1976
Water and Wastes Engineering	1964 - November, 1976
Water and Pollution Control	1960 - December, 1975
Water Resources Research	1965 - October, 1976
Water Research	1967 - November, 1976
Water Resources Bulletin	1970 - October, 1976

TABLE A-165. TEXTS AND MANUALS REVIEWED

Text	Author	Date
Sewage	Rideal	1900
Water and Wastewater Engineering	Fair, Geyer & Okun	1958
Severage and Sewage Treatment	Babbitt & Baumann	1958
Biological Waste Treatment	Eckenfelder & O'Connor	1961
Municipal & Rural Sanitation	Ehlers & Steel	1965
Industrial Water Pollution Control	Eckenfelder	1966
Water Supply Engineering	Babbitt, Doland & Cleasby	1967
Sewer Design & Construction	WPCF MOP #9	1970
Disposal of Sewage & Other Water Borne Wastes	Imhoff, et al.	1971
Water Quality & Treatment	AWWA	1971
Water Supply & Pollution Control	Clark, Viessman & Hammer	1972
Wastewater Engineering	Metcalf & Eddy	1972
Environmental Engineering & Sanitation	Salvatto	1972
Manual of Individual Water Supply Systems	EPA	1973
Manual of Septic Tank Practice	PHS	1967
Environmental Protection	Chanlett	1973
Wastewater Treatment Systems for Rural Communities	Goldstein & Moberg	1973

TABLE A-166. INDIVIDUALS AND ORGANIZATIONS FROM WHOM INFORMATION WAS REQUESTED

Individual/Organization	Description	No. of Responses*
Federal/Regional Government Agencies	Government agencies, including NASA, USEPA, U.S. Coast Guard, U.S. Navy, U.S. Army, Park Service.	61
State Environmental Agencies	The principal state environmental offices in each of the 50 states.	46
Universities	The professor in charge of the Sanitary Engineering program at each of 43 colleges and universities. Fourteen of the more well known experiment stations.	26
Establishment Associations	Associations dealing with the type of establishments being investigated.	13
Magazine Editors	The editors of 12 magazines and journals dealing with waste treatment and pollution abatement. The editors of magazines dealing with the types of establishments under investigation.	16
Miscellaneous	Attendants of various recent conferences who might have information. Equipment manufacturers and firms dealing with package treatment units and water saving devices.	42

* Responses were received to approximately 50% of the inquiries made.

ATTACHMENT D - DESCRIPTION OF LABORATORY SITE N AND THE INFLUENT WASTEWATER

PROGRAMMER CONTROL

Programmer Operations

The automation of the laboratory was accomplished by using a specially designed control system. The heart of the system was a series of 20 switch cam programmers as shown in Figure A-96. These programmers stepped forward one position when provided with an electronic signal. Operation is such that when one programmer reaches the resting position no. 20, the next programmer in the series steps to position no. 1. Since there were 5 programmers with 19 available positions (no switches can be ON at the resting position), 95 distinct steps are available. Thus, the day was divided into 96 equal 15 minute segments with all programmers going into a 15 minute rest at the end of the day.

The programmer switches could easily be programmed by sliding a portion of the adjustable cam to the ON or OFF position, providing for a great amount of flexibility in the simulation program. These switches were wired in parallel with toggle switches, which override them if a manual mode was desired.

The problem of stepping the programmers in the proper sequence and providing a resting step were handled by using a solid state counter, followed by digital logic. The logic sorted the counter signals and stepped the proper programmer corresponding to a particular number.

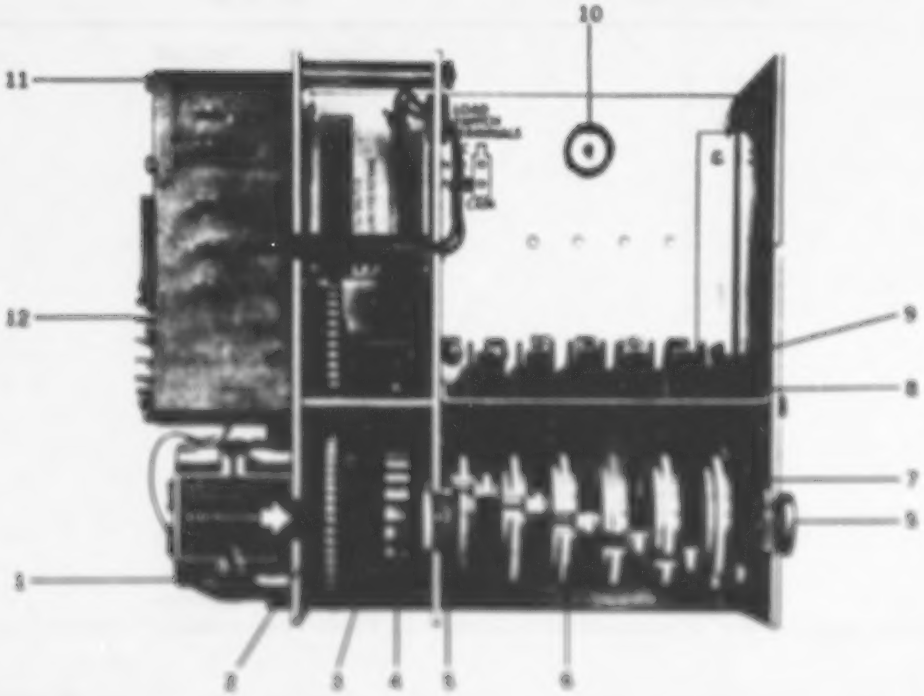
Table A-167 presents the switch assignments for the 20 programmer switches. Each switch initiated one of the subroutines. The timing of these subroutines are described below.

Program Matrix

The program matrix was an assignment of an ON or OFF position to the various switches versus a time scale (step number). Figure A-97 shows a blank matrix. The matrix was developed by choosing a desired loading schedule (Figure A-98) and staggering the starting times such that major events (all except toilet) to the various units did not overlap in a given 15 minute interval. These events were then converted to switch assignments, and the matrix was completed. A completed matrix is shown in Figure A-99 which is the matrix used during the first stages -- testing.

Simulator Control

The simulator operations were controlled by a special electronic counter similar to that used for programmer stepping. This counter counted every 7.5 seconds and is reset at the end of the 15 minute step. The timing of the opening and closing of the various valves, activation of motors and control of solenoids was accomplished by using signals from the counter. Solid state



1. Heavy-duty drive, high-torque AC pulse motor rated for continuous duty.
2. Drive gear standard, permits addition of tap switches later ... indicator strip shows step number and which tap switch circuit is made.
3. Segment indicator strip ... identifies cam segments with camshaft in or out of the unit ... for accurate programming and re-programming.
4. Control cam operates control circuit to keep motor from driving through more than one step from sustained SPST command signal.
5. Camshaft bulkheads at both ends of the camshaft hold firmly, lift out and up to release the camshaft assembly ... no tools needed.
6. Sliding-segment cams ... white segments in the white do not actuate the switches ... black segments cut out cam the latch to actuate the load switches ... 20 segments per cam, one for each step.
7. Camshaft assembly is easily removed, providing storable memory of a complete program ... keyed coupling insures proper installation.
8. Heavy duty load switches ... 10 amp SPDT Form C precision switches, independently replaceable.
9. Fully accessible terminals accept .250" push-on connectors.

Figure A-36. Cam programmer.

TABLE A-167. PROGRAMMER SWITCH ASSIGNMENTS

Switch No.	Function
1	Unit #1 Toilet
2	Unit #2 Toilet
3	Unit #3 Toilet
4	Unit #4 Toilet
5	Unit #5 Toilet
6	Blank
7	Shower
8	Bath
9	Dishwasher
10	Clothes Washer
11	Dish Rinse
12	Garbage Grinding
13	Blank
14	Blank
15	Monitor
16	Monitor
17	Monitor
18	Manifold Control
19	Manifold Control
20	Manifold Control

digital logic took these signals, along with signals from the programmer switches, and, at the proper count, performed the required function.

Distributor

Once a wastewater event was simulated, it required routing to the desired treatment unit. This was accomplished by using a specially designed distributor which is shown in Figure A-100. It consisted of a rotating pipe arm driven by a ¼ rpm gear motor. The arm stopped above one of six ports when the proper sensor was triggered (a micro-switch was tripped by the end of the arm). If no event was occurring during a 15-minute interval, the arm returned to a drain port in the event of any equipment malfunction which could add unwanted water.

The arm could be positioned via an automatic or a manual mode. The automatic mode (switches set on AUTO) took signals from switches 18, 19 and 20 of the programmer, sorted them out through digital logic, and activated the motor until the proper sensor was tripped. The proper combination of switches to be set to ON are presented in Table A-168.

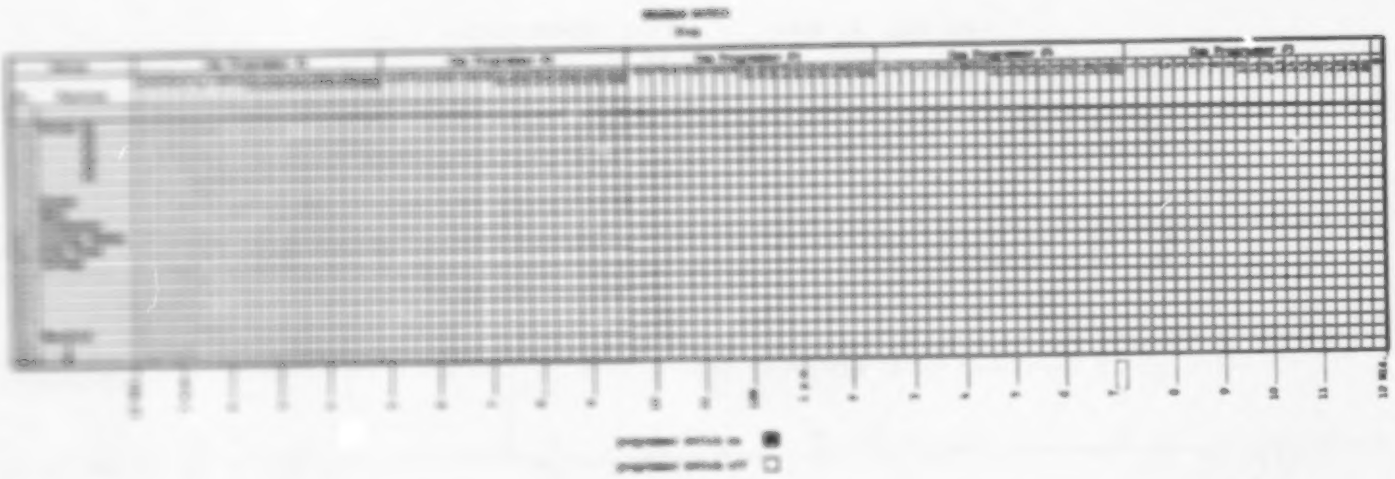


Figure A-97. Blank program matrix.

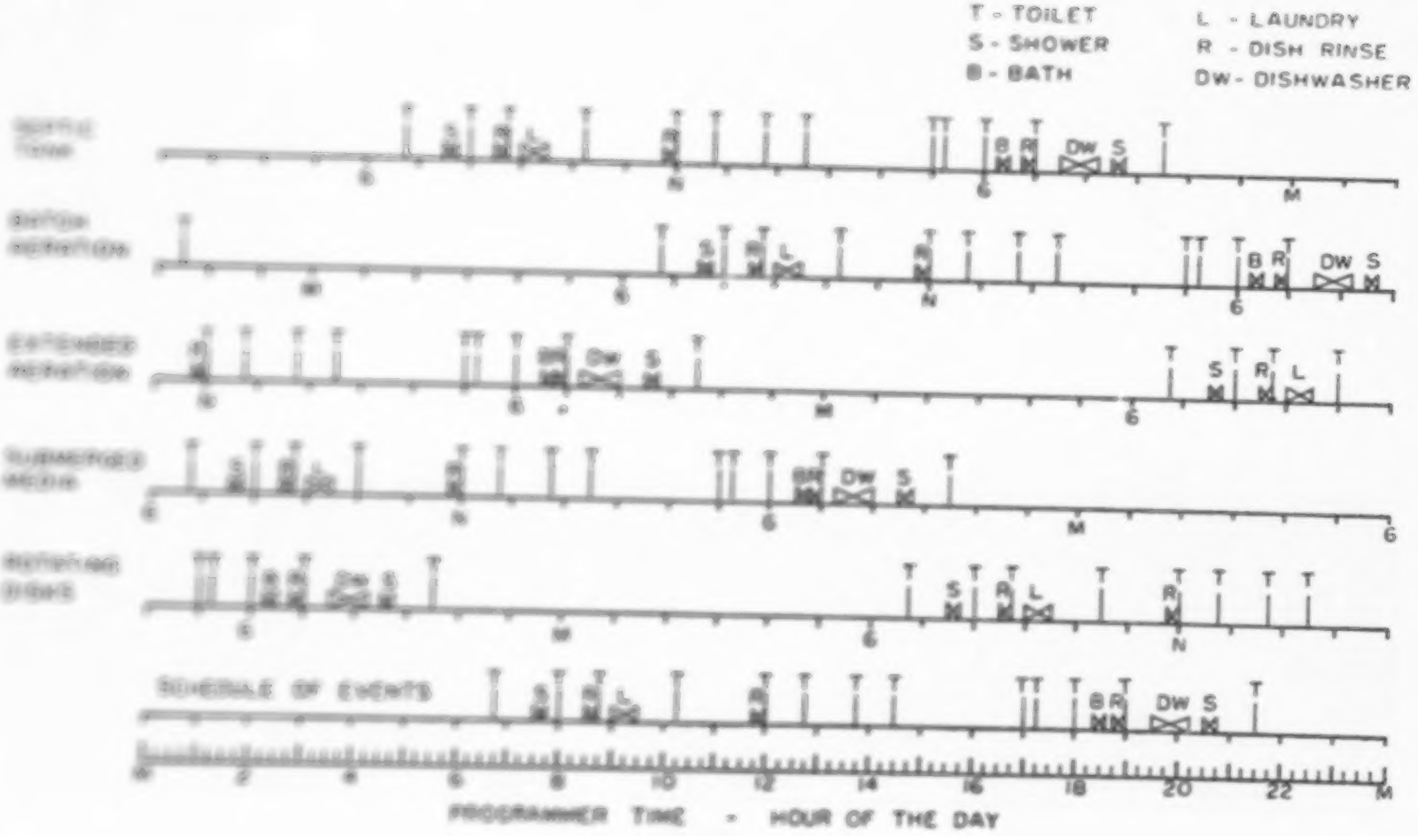


Figure A-58. Overlap of event schedules for lab units.

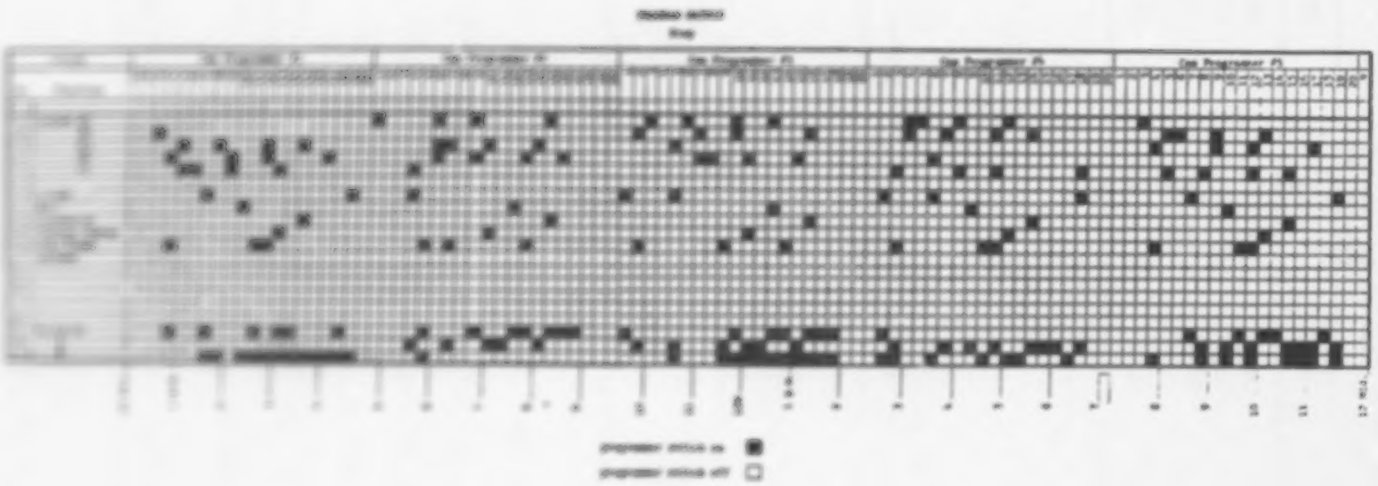


Figure A-99. Completed program matrix.

TABLE A-168. SWITCH SETTINGS

Treatment Unit	Switch No. 18	19	20
Rotating Disks	Off	Off	On
Septic Tank	Off	On	Off
Batch Aeration	Off	On	On
Extended Aeration	On	Off	Off
Submerged Filter	On	Off	On

SIMULATOR OPERATION

Laundry

The laundry was simulated by running a Maytag commercial clothes washer through its normal cycle. Switch 10 on the programmer activated this cycle. Detergent was added by the feeder shown in Figure A-101. The sequence of events was as follows:

1. Switch 10 of programmer switched ON.
2. A relay was activated to provide power to the machine timer through the machine's ticket mechanism. (Power was provided for at least 30 seconds).
3. The same relay provided power to a 2 second, time delay relay which in turn provided power to the feeder motor.
4. The feeder advanced to dump the contents of one feed container.
5. The feeder motor shut off when micro-switch was opened.
6. Power was removed from activation circuitry (Switch 10 to OFF).
7. Washing machine completed normal cycle.

The normal cycle was set at COLORS but could have been switched to any other desired setting. The machine was modified such that opening the cover did not stop the cycle. The detergent feed container held up to 450 ml of material.

In order to increase flow from the clothes washer, the cycle could be changed to PERMANENT PRESS, increasing the flow from 150 to 190 L (40 to 50 gallons).

Dishwasher

Dishwashing was simulated by running a Sears Kenmore dishwasher through its normal cycle (NORMAL WASH). Switch 3 on the programmer activates this

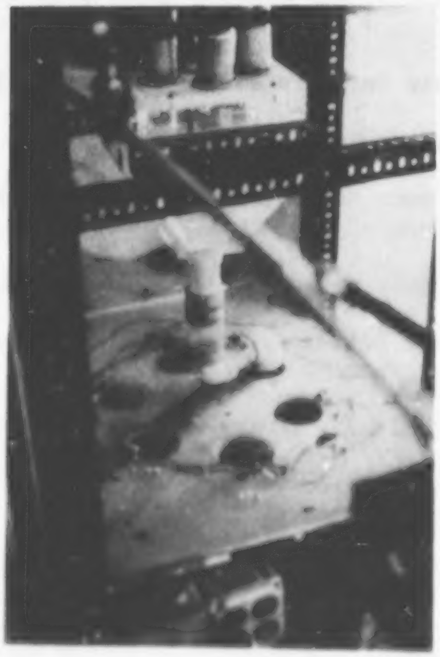


Figure A-100. Flow distribution manifold.

Figure A-101. Laundry detergent feeder.

cycle. Detergent was added to the washing machine by the feeder shown in Figure A-102.

The sequence of events for this simulator were as follows:

1. Switch 9 on programmer switched ON.
2. Relay provided power to the machine timer (power provided for a minimum of 60 seconds).
3. This same relay provided power to the feeder motor.
4. The feeder rotated to dump contents of detergent container (container held 1/4 cup of detergent).
5. The feeder automatically stopped when micro-switch was opened.
6. Power was removed from activation circuit (Switch 9 is OFF), and detergent feeder would reset for next cycle.
7. Machine completed normal cycle.

The machine was modified such that power was removed from the heating coil for the last half of the machine cycle by using the power which normally was used by the blower.

Dish Rinsing

The dish rinse was simulated by a specially designed apparatus shown in Figure A-103. The electronics were activated by switch 11 of the programmer. A blended slurry of assorted food materials was automatically added along with a metered amount of hot water in the following sequence:

1. Switch 11 of programmer switched ON.
2. Electronic control turned mixing motor on.
3. Air cylinder retracted allowing measuring chamber to fill with slurry.
4. Air cylinder extended, isolating measuring chamber from slurry reservoir. Measuring chamber empties.
5. Mixing motor turned off.
6. Electronic control opened solenoid valve to wash slurry into distributor. Time was variable from 0 to 180 seconds and controlled by a time delay relay.
7. Valve closed.
8. Switch 11 of programmer switched OFF.

Figure A-102. Dishwasher detergent feeder.

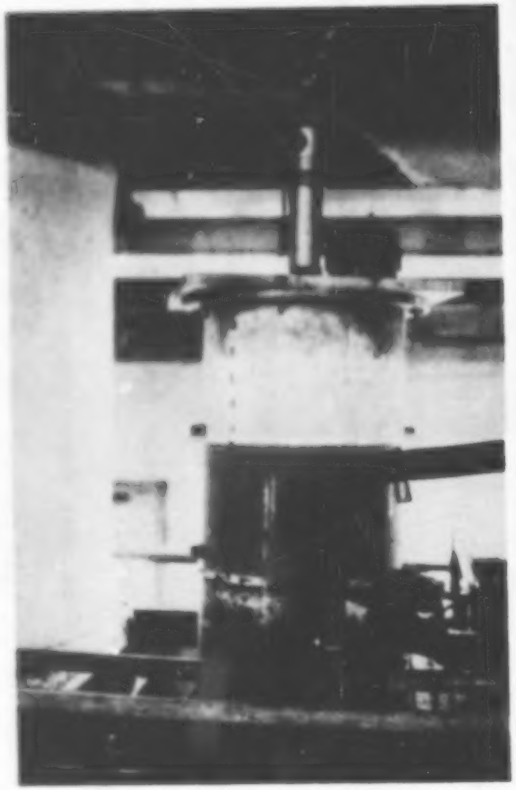


Figure A-103. Dish rinse feeder.

The feed slurry reservoir held up to 11 liters of slurry which was blended manually in a commercial size Waring blender. The measuring chamber was about 200 mL in size.

Bath

The bath was simulated by a specially designed simulator shown in Figure A-104. Liquid hand soap was added automatically with a piston feeder pump. The sequence of events were as follows:

1. Switch 8 of the programmer switched ON.
2. The electric controlled ball valve opened.
3. Power was provided to soap feeder pump for 1 minute to pump a variable volume of soap (0-100 mL).
4. Power was removed from soap feeder pump.
5. Electric controlled ball valve closed after tank has emptied.
6. The solenoid fill valve opened to fill barrel.
7. Solenoid fill valve closed when water level reaches PVC float.
8. Switch 8 of programmer switched OFF.

The rate at which the barrel emptied was controlled by opening or closing a gate valve following the ball valve. The total volume was controlled by adjusting the level of the PVC float.

Shower

The shower was simulated by a timer controlled solenoid valve. Liquid soap was added automatically with a piston feed pump. The system is shown in Figure A-105. The sequence of events were as follows:

1. Switch 7 of programmer switched ON.
2. Timer solenoid valve opened allowing a flow rate of 11.3 L/m (3 gpm).
3. Power was provided to soap feeder pump for 1 minute to pump a variable volume of soap (0-100 mL).
4. Power was removed from soap feeder pump.
5. Timer closed solenoid valve after selected time (adjustable from 0 to 15 minutes).
6. Switch 7 of programmer switched OFF.

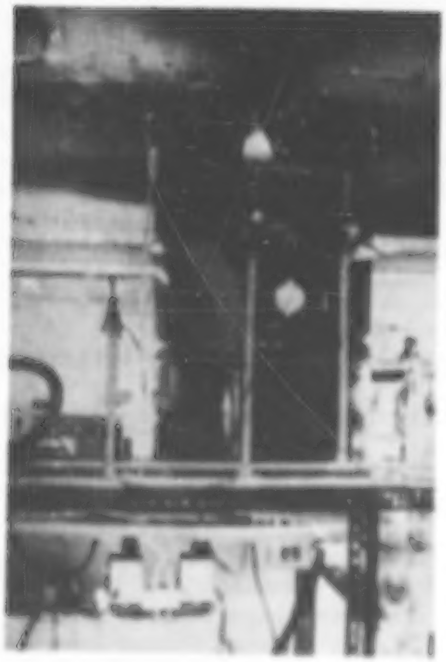


Figure A-104. Bath event simulator

Figure A-105. Bath and shower soap feeder

Toilet

The toilet was simulated by a specially designed system shown which could be broken down into 3 distinct functions: urine addition, feces addition, and water flush. The urine solution was a specially mixed solution of salts, metered to each unit via a constant volume measuring chamber. Provisions were made to add feces manually. The flushing water comes from a standard toilet tank which was automatically flushed. A water cut-off valve was installed in the water line in the event that the toilet ball could not seat properly. Each treatment unit was equipped with its own simulator (5 total). The sequence of events were as follows:

1. Switch 1, 2, 3, 4, or 5 of programmer switched ON.
2. Upper solenoid valve on urine measuring chamber opened allowing chamber to fill.
3. Upper solenoid valve closed.
4. Bottom solenoid valve and water cut-off valve opened simultaneously.
5. Solenoid was activated to flush toilet tank.
6. Solenoid was deactivated.
7. Bottom solenoid valve and water cut-off valve closed (after 2 minutes to allow tank to fill).
8. Switch 1, 2, 3, 4, or 5 of programmer switched OFF.

With the manual addition of feces (dog manure) a certain amount of toilet paper was added once per day.

INFLUENT WASTEWATER CHARACTERISTICS

The laboratory units were fed wastewater that was generated by the equipment described previously. The purpose of this discussion is to outline the characteristics of this simulated wastewater and to summarize (by months) their average values.

Toilet Simulation (Urine, Feces and Paper)

Urine--

Urine is simulated by addition of a salt solution containing the following:

Urea = 8.67 kg
Na_3PO_4 = 0.83 kg
Na_2SO_4 = 0.75 kg
NaCl = 0.16 kg

KCl = 1.00 kg
NH_4Cl = 0.44 kg
HCl = 0.30 l
Water to make 200 liters

This solution had the following characteristics in grams per liter:

	BOD_5	COD	TS	TSS	TN	TP
g/L	.055	4.7	29	.038	8.0	1.3

and the following monthly utilization:

Jan 75	3.86	Sept 75	4.52	L/unit/day
Feb	1.46	Oct	3.82	
Mar	3.47	Nov	4.23	
Apr	3.48	Dec	4.20	
May	3.88	Jan 76	4.00	
June	4.01	Feb	3.99	
July	3.99	Mar	4.15	
Aug	4.21			

Feces--

Hog manure is added manually to each of the units daily. Several analyses of the manure yielded the following average characteristics:

	BOD_5	COD	TS	TSS	TN	TP
g/g of feces	.057	.21	.20	.16	.015	.0098

The average daily addition was:

Jan 75	432	Sept 75	680	g/unit/day
Feb	411	Oct	680	
Mar	453	Nov	566	
Apr	453	Dec	680	
May	453	Jan 76	549	
June	453	Feb	680	
July	402	Mar	636	
Aug	402			

Paper--

Nineteen grams of toilet paper were added manually to each unit every day of the test period. This quantity of paper had the following characteristics:

	BOD_5	COD	TS	TSS	TN	TP
g/unit/day	4.7	18	19	19	0	0

Laundry

Detergent was added to each laundry event. There was one event per day. Tide detergent had the following characteristics:

	BOD_5	COD	TS	TSS	TN	TP
g/g of detergent	.21	.44	.86	.21	.004	.086

and the following daily utilization:

Jan 75	147	Sept 75	150 g/unit/day
Feb	90	Oct	145
Mar	90	Nov	111
Apr	129	Dec	148
May	118	Jan 76	150
June	142	Feb	83
July	144	Mar	129
Aug	150		

Bath/Shower

Liquid hand soap added to each of the 3 daily bath/shower events had the following characteristics:

	BOD_5	COD	TS	TSS	TN	TP
g/mL of soap	.75	1.25	.33	.46	.01	.008

The following are the average daily utilization:

Jan 75	106	Sept 75	45 mL/unit/day
Feb	53	Oct	44
Mar	50	Nov	47
Apr	73	Dec	50
May	63	Jan 76	63
June	62	Feb	90
July	51	Mar	17
Aug	59		

Kitchen

Dish Rinse--

A specially blended slurry containing the following ingredients was fed into the units after March 16, 1975:

2 cans beef stew (5 lb),
6 cans chicken noodle soup (4 lb),
6 cans vegetable beef soup (4 lb),
200 mL cooking oil,
200 mL soap, and
Water to make 9.5 liters of slurry.

Before this date a somewhat weaker slurry was added. The characteristics of these slurries are:

	BOD_5	COD	TS	TSS	TN	TP	
g/mL of slurry	.044	.085	.062	.036	.0024	.001	to 3/16/75
	.060	.113	.105	.056	.0057	.001	after 3/16/75

The following amounts were added to each unit on an average basis:

Jan 75	606	Sept 75	660 mL/unit/day
Feb	654	Oct	574
Mar	590	Nov	446
Apr	540	Dec	599
May	580	Jan 76	621
June	581	Feb	517
July	625	Mar	686
Aug	600		

Dishwashing--

Detergent is added to each wastewater event and has the following characteristics:

	BOD_5	COD	TS	TSS	TN	TP
g/g of detergent	.004	.047	.73	.22	.0023	.136

The average daily use by month was:

Jan 75	55	Sept 75	51 g/unit/day
Feb	55	Oct	51
Mar	70	Nov	60
Apr	27	Dec	58
May	27	Jan 76	64
June	38	Feb	86
July	20	Mar	73
Aug	51		

Influent Wastewater Summary

Table A-169 was prepared by combining each event waste characteristics and relative flow rate. These values are summed to get total grams of characteristic per unit per day (e.g. g BOD_5/unit/day). The total mass of the various parameters are also calculated in order to determine the average values for the test period.

Grease

Grease determinations were made on several samples. The grease content of the influent waste was about 120 mg/L of which 50% was contributed by the feces and the dish rinse slurry.

TABLE A-169. TEST PERIOD INFLUENT WASTE CHARACTERIZATION (1/1/75 - 3/31/76)

Month	Flow*	BOD_5	COD	TS	TSS	TN	TP
Jan 75	820	167	378	456	202	41	31
Feb	740	116	275	318	164	20	22
Mar	740	113	285	392	170	37	27
15th	740	135	328	469	194	39	32
Apr	740	145	322	423	188	39	25
May	740	138	329	426	184	42	24
June	740	142	339	457	191	52	28
July	740	134	320	437	177	43	25
Aug	740	140	332	469	187	48	30
Sept	740	149	381	536	228	51	34
Oct	740	142	365	503	222	45	32
Nov	640	123	300	457	193	46	30
Dec	740	148	378	525	228	48	34
Jan 76	740	152	369	505	215	45	33
Feb	740	160	391	489	235	46	32
Mar	716	122	330	508	210	48	33
Totals	340 m^3	63.9 kg	156 kg	211 kg	91.4 kg	19.9 kg	14.0 kg

* All entries in g/unit/day, except flow in L/d.

APPENDIX B

SOIL ABSORPTION OF WASTEWATER EFFLUENTS

PART 1

SITE CHARACTERIZATION FOR WASTEWATER ABSORPTION

A successful wastewater treatment system produces an effluent which is compatible with the environment to which it is discharged. The environment, of course, is part of the system providing the final treatment necessary before the water is of sufficient quality for reuse. If the pollutant load received by the environment is too great, the environment will not break down and recycle the pollutants rapidly enough. The result is that the pollutants will accumulate, leading to failure of the system.

The maximum permissible limits of pollutants that can be discharged to the environment vary with the type of pollutant and the local environment into which they are discharged. To properly design a treatment system, therefore, it is necessary to evaluate the physical characteristics of the local environment where discharge of the partially treated wastewater is to be made. Each site has its own characteristics that limit its potential as a treatment medium.

Proper evaluation of the receiving environment becomes particularly critical where onsite wastewater treatment systems are necessary. Onsite systems lack the advantage of institutional control of central sewerage where wastes can be collected and conveyed to a treatment plant located at a site which is selected for its suitability to receive the pretreated wastes. Instead, they must be located near the point of waste generation and local environmental conditions are often less than optimal.

Traditionally, the septic tank-soil absorption system has been used to provide onsite treatment and disposal of liquid wastes. Soils are very effective biological and physical filters which break down organic and other chemical substances as well as remove pathogenic organisms and viruses. However, not all soils and the site characteristics with which they are associated, are equally effective in providing absorption and purification over a reasonable lifetime. Therefore, site and soil characterization is necessary to select a suitable system design.

The factors that influence the design and operation of onsite absorption systems include the hydraulic conductivity, depth of soil over zones of saturation or bedrock, the slope and topographic position, and the site's management history. Proper site selection requires a knowledge of how these factors influence system operation and the methods of successfully determining them.

FACTORS INFLUENCING SITE SUITABILITY FOR LIQUID WASTE DISPOSAL

Soil Hydraulic Conductivity

The rate of acceptance of liquid by a soil absorption system will be limited by the hydraulic conductivity of the soil and the resistance of any impeding layer at the infiltrative surface. If the soil cannot accept the water at the rate it is applied because the soil is too slowly permeable and/or the clogging mat is too resistant, failure will occur.

Direct measurement of the hydraulic conductivity at the moisture potential expected for the system design gives a loading rate for sizing. Different methods of measuring or estimating the hydraulic properties of soils have been used. Some of these are based on the known physical character of liquid in soil materials while others are empirical.

Physical Characterization of Liquid in Soil Materials--

Soil wetness refers solely to the total amount of liquid in a soil sample. Soil wetness can be expressed in two ways:

$$\text{Percentage by weight: } \theta_w = \frac{100 \cdot M_w}{M_s}$$

Where, M_w = mass of water (g)

M_s = oven dry weight of soil (g) and

$$\text{Percentage by volume: } \theta_v = \frac{100 \cdot V_w}{V_s + V_a + V_w}$$

where, V_w = volume of water (cm^3)

V_s = volume of soil solids (cm^3) and

V_a = volume of soil air (cm^3)

These two characteristics are interrelated as follows:

$$v = \frac{\theta_w \cdot B.D.(dry)}{\rho_w}$$

Where, ρ_w = density of water (g/cm^3)

B.D. = the bulk density (g/cm^3)

Soil volume is a relevant factor in several soil characteristics. This volume may change when dry soil is wetted. Most soil materials containing clay will expand upon wetting, which is mainly due to the mineralogical and chemical nature of the clay minerals and to chemical characteristics of the wetting liquid. Thus, it follows that bulk density and θ_v values will be affected. In addition, it is important to ascertain the distribution of water in the soil at different moisture contents and to understand the natural laws that govern it. As the moisture content of a soil sample decreases, water leaves the larger soil pores but remains in the finer ones. This can be explained by considering the basic phenomena of liquid surface tension and capillarity. Surface tension occurs at the interface of a liquid and a gas. Molecules in the liquid attract each other from all sides. At the surface areas the molecules are attracted into the denser liquid phase by a force greater than the force attracting them into the gaseous phase. The resulting force draws the surface molecules inward, which results in a tendency for liquid to contract. Surface tension has the dimension of dynes/cm. Increased salt concentrations tend to increase the surface tension of water, whereas organic solubles like detergents tend to decrease it. Capillarity refers to the well-known phenomenon of the rise of water into a capillary tube inserted in water, due to its surface tension (Figure B-1). The finer the tube, the higher the capillary rise and the greater the negative pressures below the water meniscus in the tube. This negative pressure (p) is a result of the curvature of the meniscus, which increases as tubes become smaller, and can be calculated (in dynes/cm^2) as follows (assuming that the contact angle between water and tube is zero):

$$p = \frac{2\delta}{r}$$

Where, δ = surface tension of the water (dynes/cm)

r = radius of the capillary (cm)

The height of capillary rise (cm) is

$$h = \frac{2\delta}{\rho g r}$$

Where, g = gravitational constant (cm/sec^2)

Pictured as a continuous graph in Figure B-1 capillary radius is related to corresponding pressures. This negative pressure below the meniscus in the water can thus be expressed in terms of the height of the column of water (cm) that can be "pulled" from a cup of water by the capillary tube. For example, a cylindrical pore diameter of 100 microns corresponds with a relatively low capillary rise of 28 cm water (pressure below meniscus = -28 cm water), while a diameter of 30 microns corresponds with a relatively high rise of 103 cm (pressure -103 cm water). These figures imply that it takes a greater force (more energy) to remove water from a small pore than from a large one.

To represent the porosity of a certain soil material as a bundle of capillaries with a characteristic size range is, of course, a simplified model since real pores in the soil have a much more complex configuration, with varying sizes and discontinuities. This representation can nevertheless be helpful to visualize the energy condition of water in soil and flow

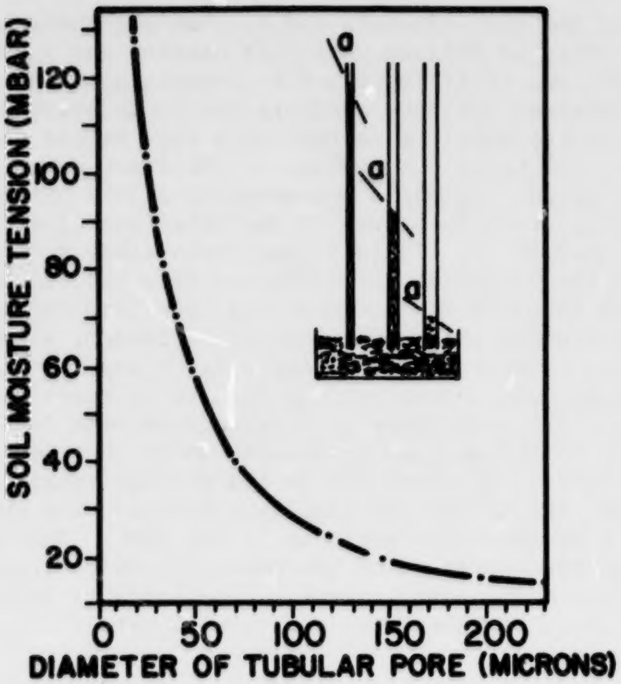


Figure B-1. Graphical expression of the relationship between tubular pore size and corresponding soil moisture tension (Bouma et al., 1972).

phenomena, particularly when soil moisture contents are relatively close to saturation.

Soil moisture potential--Water moves from points where it has a higher potential energy status to points where it has a lower potential status. The energy status is referred to as the "water potentia," a central concept in soil physics. A thorough and clear discussion of this concept has been given by Rose (1966) and only a brief summary will be presented here. The total potential or energy per unit quantity of water (ψ_t) is defined as the mechanical work required to transfer a unit quantity (e.g. unit mass weight or volume) of water from a standard reference state ($\psi = 0$) to one where the potential has the defined value. The total potential (ψ_t) of water (which in the following will be expressed in terms of energy per unit of weight because this results in the simplest expression) is composed of several components, which will be discussed separately.

Pressure potential (ψ_p)--Water in unsaturated soil occurs only in the finer pores because the total amount of available water is insufficient to fill all the pores and the smallest pores can "pull" strongest. The pressure in the water is then less than that of the local atmosphere. As the available amount of water decreases, the moisture pressure becomes more negative. It is convenient, but not necessary, to refer to a negative (less than atmospheric) pressure as a "tension" or "suction." A tension or suction of, for example, 30 cm water represents a soil water pressure of -30 cm water. Also, low "tensions" or "suctions" are equivalent to relatively low "pressures."

The pressure potential in unsaturated soil is referred to as the matric or capillary potential (ψ_m). The matric potential is used extensively in the following sections on hydraulic conductivity. Expressed per unit weight, the dimension of the matric potential becomes

$$\psi_p/\rho g = \frac{g \cdot cm \cdot t^{-2} \cdot cm^{-2}}{g \cdot cm^{-3} \cdot cm \cdot t^{-2}} = cm.$$

This notation is more convenient than others expressing the potential per unit mass or volume.

The matric potential, which can be measured by tensiometry is the most important component of the pressure potential in most cases because soils above the water table are usually unsaturated. The soil water is at a pressure higher than one atmosphere if submerged beneath a free water surface. The potential associated with this has been called the submergence potential (S) by Rose (1966). The submergence and matric potentials are mutually exclusive possibilities; if either of them is nonzero, the other must be zero.

Gravitational potential (ψ_g)--The gravitational potential is due to the attraction of every body on the earth's surface toward the center of the earth by a gravitational force equal to the weight of the body. To raise this body against this attraction, work must be done, and this work is stored by the raised body in the form of gravitational potential energy (Z), which is determined at each point by the elevation of the point relative to some arbitrary reference level. Therefore

$$\psi_g = mgZ$$

Where ψ_g = the gravitational potential energy of a mass m of water at a height Z above a reference and

g = acceleration of gravity

This potential, expressed per unit weight, becomes $\psi_g = Z$ (in cm).

Osmotic (ψ_o) *and overburden potential* (ψ_Ω)--The osmotic potential describes the effect of solutes on the total potential of soil water and is important for the study of water movement into and through plant roots and for studies of evaporation and vapor movement. The overburden potential is important when soils are free to move and its weight becomes involved as a force acting on the water. The osmotic potential and overburden potentials will not be discussed further here, since they do not significantly affect the mass movement of water through soil between tensions of 0 and 100 cm under the conditions of most interest in the context of this review.

Total potential (ψ_t)--The total potential of soil water at any place in the soil is thus equal to the sum of the major component-potentials ψ_p, ψ_g, ψ_o, and ψ_Ω. Theory of water flow uses the hydraulic potential (cm) (or head), which is the sum of pressure and gravitational potentials previously defined. It is common to refer to the hydraulic potential in terms of the "hydraulic head" (H) (cm). Therefore:

$$\psi_t = \psi_p \text{ (or } \psi_m) + \psi_g + \psi_o + \psi_\Omega$$

Of major concern here is

$$\psi_t = \psi_p \text{ (or } \psi_m) + \psi_g$$

Moisture retention in soil--At zero tension all pores in the soil are filled with water (assuming that isolated air pockets do not exist). With increasing soil moisture tension, progressively smaller pores will empty as the capillary force they can exercise becomes insufficient to retain water against the tension applied. The rate of decrease of water content in a soil sample upon increasing tension is a function of the pore-size distribution for each soil material. This simplified discussion assumes that the soil matrix is rigid and that water extraction does not result in soil shrinkage, thereby releasing water without creating empty voids. This is, however, an important process in clayey soils.

Techniques are available to experimentally determine the so-called soil moisture retention curve, which gives the water content of the soil at any given tension (see Fig. B-2). The simple apparatus in Figure B-3 can be used to establish the moisture retention curve. The saturated soil sample is placed on a porous disk inside the Buchner-type cup. A tension is applied by adjusting the point of water outflow at a specific level below the plate. A detailed discussion of this and another important method can be found in the literature (Bouma et al., 1974a).

Figure B-2 shows moisture retention curves for a sand (C horizon of Plainfield loamy sand), a sandy loam (IIC of Batavia silt loam), a silt loam (B2 of Batavia silt loam), and a clay (B2 of Hibbing loam), demonstrating the effect of their different pore types. The sand has many relatively large pores that drain at relatively low tensions or pressures, whereas the more clayey soils release only a small volume of water because most of it is strongly retained in fine pores. The sandy loam has more coarse pores than the clay soil.

Moisture contents in a soil sample are different at corresponding tensions, depending on whether the moisture content was reached by removing

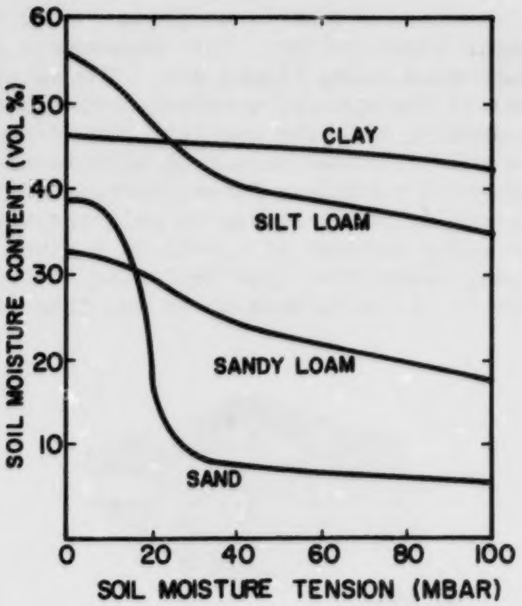


Figure B-2. Soil moisture retention curves, relating soil moisture content to moisture tension, for four different soil materials (Bouma et al., 1972).

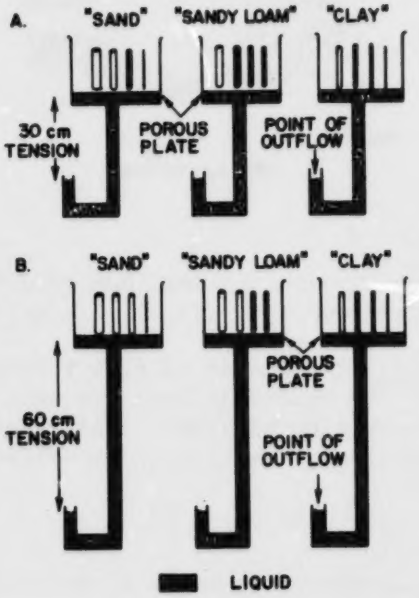
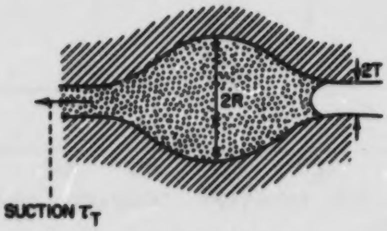
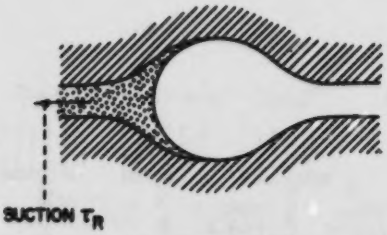

A.
"SAND"
"SANDY LOAM"
"CLAY"
30 cm
TENSION
POROUS
PLATE
POINT OF
OUTFLOW
B.
"SAND"
"SANDY LOAM"
"CLAY"
60 cm
TENSION
POROUS
PLATE
POINT OF
OUTFLOW
■ LIQUID

Figure B-3. Moisture retention in three schematic soil materials at tensions of 30 and 60 cm (Bouma et al., 1974a).

water from an initially wetter sample (desorption) or by adding water to an initially drier sample (adsorption). This phenomenon is referred to as hysteresis and is illustrated using Figure B-4. The water-filled void (top) will drain (desorption) if the applied tension exceeds the relatively large capillary force corresponding with the smallest pore diameter (2r) in the system. An air-filled void (bottom) will fill with water (adsorption) as soon as the relatively small capillary force, corresponding with the largest pore diameter (2R), is sufficiently strong to pull the water in. This comparison shows that the water content of a soil at a given soil moisture tension will be greater following desorption than following adsorption. It takes more energy to get water out of the soil once it is in, than to get it back in.



(a) DESORPTION



(b) ADSORPTION

Figure B-4. Cross section through an idealized void illustrating the hysteresis phenomenon (Bouma et al., 1974a).

Water movement in soil--The amount of flow through a soil sample is proportionate to the drop of the hydraulic head per unit distance (hydraulic gradient = $\Delta H/L$) in the direction of flow. This, basically is Darcy's law as stated for a one-dimensional, steady-state condition of flow:

$$V = K \cdot \frac{\Delta H}{L}$$

where K is the hydraulic conductivity and V is the flux.

The flux (V) is measured across unit cross-sectional area. Part of that area (at least 40%) is occupied by the solid phase, which implies that the real velocity of flow in the soil pores is larger than V. If the soil were composed of simple capillary tubes of specific sizes, calculations of the real flow velocity in those pores would be easy. However, pores vary in shape, width, and direction, and the actual flow velocity in the soil pores is variable. At best, therefore, one can refer to some "average" velocity (v') that can be calculated on the basis of the water-filled porosity at each tension.

$$v' = \frac{V}{\varepsilon_w}$$

where ε_w is the water-filled porosity, as derived from the moisture retention curve. At unit hydraulic gradient ($\Delta H/L = 1$), we find

$$v' = \frac{K}{\varepsilon_w}$$

Using these relationships, travel times at different moisture contents during steady-state flow can be estimated for different soil horizons if a K curve is available. According to the above equation, flow rates in a given soil material at a certain moisture content can vary considerably with varying hydraulic gradient. The hydraulic conductivity (K), however, is defined as the flux at unit gradient, and can, therefore, be considered a characteristic value for the soil. K curves of different soil materials vary widely due to different pore size distributions in the soils.

Physical equations have been developed to express flow rates based on pore size at a given hydraulic gradient (Childs, 1969). The equations shown in Figure B-5 express the flow rate Q/t ($cm^3/cm^2/sec$) as a function of the density of water ρ (gr/cm^3), gravitational constant g (cm/sec^2), viscosity η (dyne/cm), hydraulic gradient symbolized by grad ϕ (cm/cm) and the pore radius r (cm) or width D (cm). These equations are graphically expressed in Figure B-6, demonstrating the significant effect of pore size on flow rates.

The dominant effect of pore sizes on permeability is evident when comparing K values of a soil material measured at different degrees of saturation. Unsaturated soil below an infiltrating surface may occur because of a physical barrier to flow at the infiltrative surface or an application rate of liquid which is lower than the saturated hydraulic conductivity of the soil. We may assume three different soil materials, with pore size distributions schematically represented in Figure B-7. The uppermost "soil" is coarse porous material (like a sand) and the lowest one is fine porous material (like a clay). Without a physical barrier ("crust") at the soil surface and with a sufficient supply of water, all pores are water-filled and each will conduct water downward as a result of the potential gradient of 1 cm/cm due to gravity. The larger pores will conduct much more water than the smaller ones. If a weak crust forms over the tops of the tubes, pores will fill with water only if the capillary force they can exercise is strong enough to "pull" the water through the crust. The larger the pore, the smaller the capillary force that can be

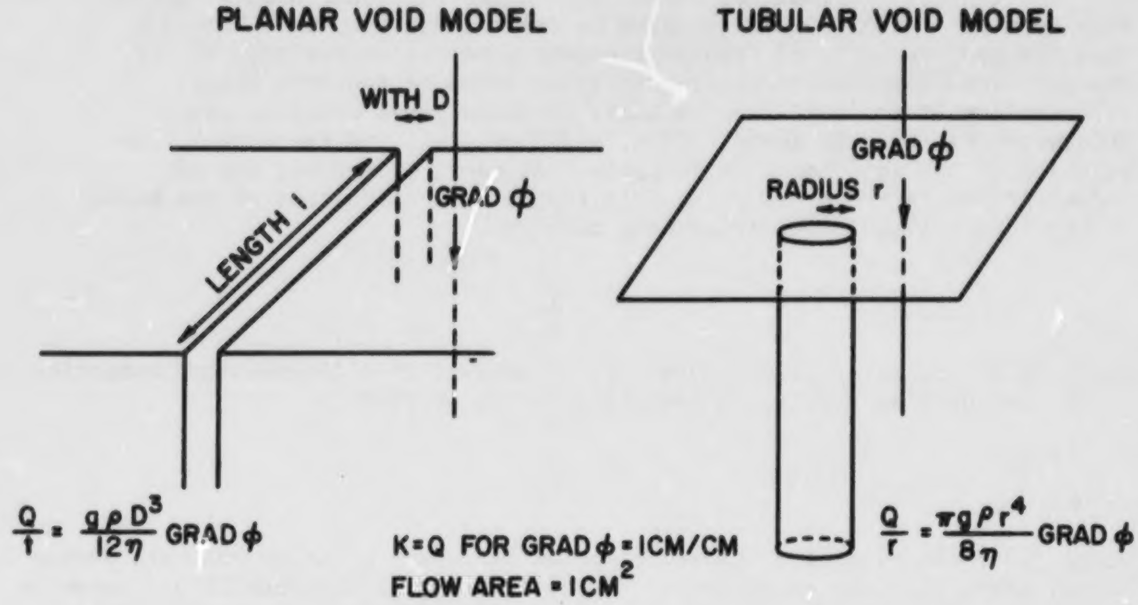

PLANAR VOID MODEL
TUBULAR VOID MODEL
WITH D
LENGTH l
GRAD φ
GRAD φ
RADIUS r
Q/l = gρD³/12η GRAD φ
K = Q FOR GRAD φ = 1CM/CM
FLOW AREA = 1CM²
Q/r = πgρr⁴/8η GRAD φ

Figure B-5. Relationships between sizes of tubular and planar voids and flow rates at defined hydraulic gradients (Bouma et al., 1972).

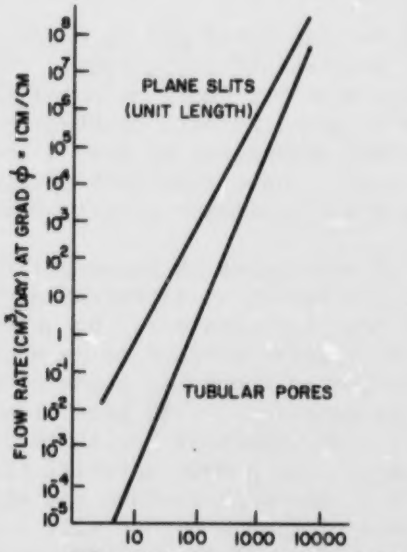


Figure B-6. Graphical expression of flow rates through tubular or planar voids as a function of pore size at a hydraulic gradient of 1 cm/cm (Bouma et al., 1972).

exercised. Therefore, larger pores will empty first at increasing crust resistance, creating unsaturated soil and soil moisture tensions which, in turn, leads to a strong reduction in the hydraulic conductivity of the soil.

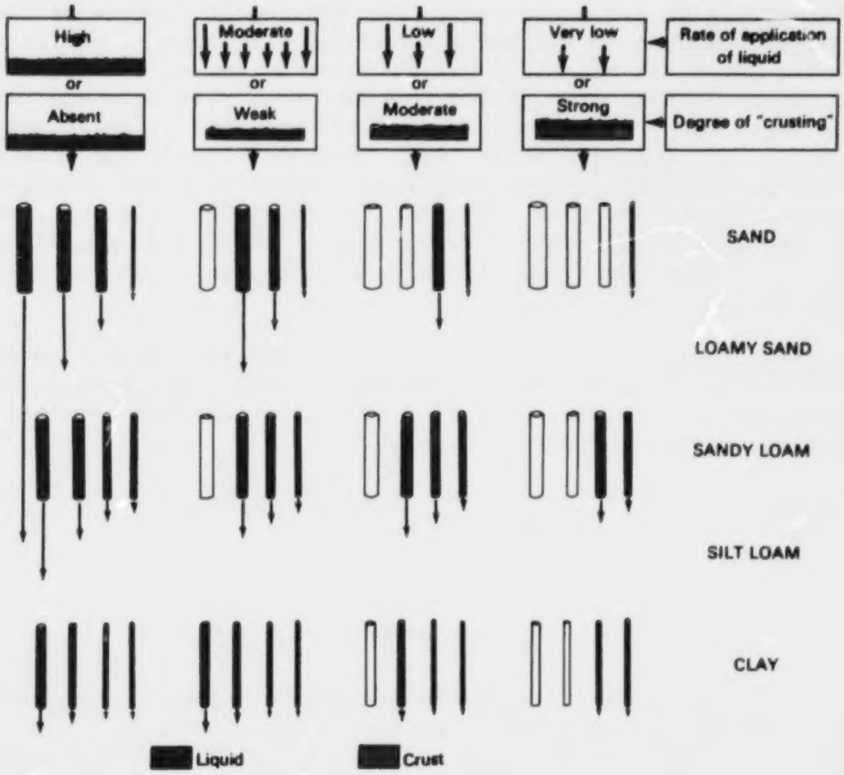

High
Moderate
Low
Very low
Rate of application of liquid
or
or
or
or
Absent
Weak
Moderate
Strong
Degree of "crusting"
SAND
LOAMY SAND
SANDY LOAM
SILT LOAM
CLAY
Liquid
Crust

Figure B-7. Schematic diagram showing the effect of increasing the degree of crusting or decreasing the rate of application of liquid on the rate of percolation through three "soil materials" (Bouma et al., 1972).

With no crusts present, unsaturated soil conditions can occur when the rate of application of water to the capillary system is reduced. With abundant supply, all pores are filled. As this supply (which is supposed to be divided evenly over the infiltrating pore system) is decreased, there is not enough water to keep all pores filled during the downward movement of the water. Assuming that the pores are horizontally interconnected, larger pores will empty first, as they conduct most liquid, while at the same time they exercise only relatively small capillary forces. In this system a certain size of pore can be filled with water only if smaller pores have an insufficient capacity to conduct away the applied water.

The rate of reduction in K upon desaturation and increasing soil moisture tension is characteristic for the pore size distribution of the soil material. Coarse porous soils have a relatively high saturated hydraulic conductivity (K_{sat}), but K drops rapidly with increasing tension. Fine porous soils have a relatively low K_{sat}, but K decreases more slowly upon increasing tension. Experimental curves, determined with the crust test show such patterns for natural soil. Figure B-8 shows curves for the sand C horizon of the Plainfield loamy sand, the sandy loam IIC horizon and the silt loam B2 horizon of the Hibbing loam. Moisture retention curves for the same soil horizons were presented in Figure B-2. The curves for the pedal silt loam and clay horizons demonstrate the physical effect of the occurrence of relatively large cracks and root and worm channels. Soil structure inside the peds is very fine porous and these fine pores contribute little to flow. The large pores between peds and root and worm channels give relatively high K_{sat} values (140 cm/day for the silt loam), but these pores are not filled with water at low tensions and K values drop very strongly between saturation and 20 cm tension (1.5 cm/day for the silt loam).

Flow into crusted soil--A special case of the two-layer flow system occurs when very thin layers with different hydraulic properties occur in a flow system. An example will be discussed concerning thin barriers (crusts) on top of infiltrative surfaces (Hillel, 1971).

Assuming steady infiltration the flux through the crust (q_b) should be equal to the flux in the subcrust soil (q_s).

$$q_b = q_s \text{ or } K_b \left(\frac{dH}{dZ}\right)_b = K_s \left(\frac{dH}{dZ}\right)_s$$

where K_b and K_s are hydraulic conductivities of the barrier and the underlying soil with dH/dZ the hydraulic head gradient in each material. The hydraulic head gradient will be approximately unity in the soil at steady infiltration (Baver et al., 1972). Assuming flow in the soil to result only from gravitational forces:

$$q = K_{s(\psi_m)} = K_b \cdot \frac{H_o + \psi_m + Z_b}{Z_b} \quad \text{or}$$

$$\frac{K_{s(\psi_m)}}{H_o + \psi_m + Z_b} = \frac{K_b}{Z_b} = \frac{1}{R_b}$$

where $K_{s(\psi_m)}$ is the unsaturated K value of the soil at a moisture tension of ψ_m cm, H is the positive hydraulic head on top of the barrier exerted by the depth of ponded liquid, Z is the thickness of the barrier, and R_b =

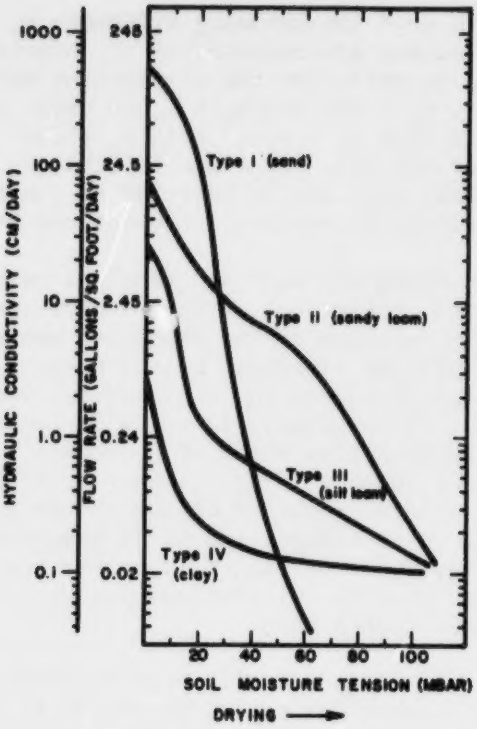


Figure B-8. Hydraulic conductivity (K) as a function of soil moisture tension measured *in situ* with the crust-text procedure (Bouma, 1975).

hydraulic resistance of the barrier, R_b can be determined from Darcy's law as applied to the barrier:

$$q = K_b \cdot \frac{\Delta H}{Z_b} \quad \text{or} \quad q = \frac{K_b}{Z_b} \cdot H = \frac{\Delta H}{R_b}$$

ΔH can be determined as: $H_o + \psi_m + Z_b$. When the flux q is known from the measured tension and the K curve, R_b can be determined from this equation, K_b can be calculated if barrier thickness (Z_b) is known; otherwise use of R_b is most appropriate.

The hydraulic effects of barriers can be predicted using the above equations when K curves are available for the soils below the barriers and when R_b values are known for the barriers themselves (Bouma, 1975). This is illustrated in Figure B-9 where K curves, measured *in situ* with the crust test are shown for a sand, a sandy loam, a silt loam and a clay. The other cruves were derived from the above equation assuming different R_b and H_o values, and a value of 2 cm for Z_b. The curves are composed of all points where the relationship between $K_s(\psi_m)$ and ψ_m is valid for the assumptions made. Curves were drawn in Figure B-9 for $R_b = 5$, $R_b = 100$ and $R_b = 1,000$ days, combined with $H_o = 5$, $H_o = 30$ and $H_o = 60$ cm (for $R_b = 1,000$ only $H_o = 5$). Points where both types of curves cross represent the only hydraulic conditions, in terms of tensions below barriers and flow rates, that can be expected at the specified H_o, Z_b and R_b values. Some conclusions of practical interest can be drawn from Figure B-9:

1) Infiltration rates decrease and tensions below the barrier increase as the resistance of the barrier increases. The effects are a function of the capillary properties of the underlying soil, as expressed by the K curve. For example, a barrier of $R_b = 5$ days ($H_o = 5$) induces tensions of 35 cm (sandy loam), 28 cm (sand), 13 cm (silt loam) and 3 cm (clay) with corresponding flow rates of 8 cm/day; 7 cm/day; 4 cm/day and 1.8 cm/day, respectively. A barrier of $R_b = 1,000$ days ($H_o = 5$ cm) induces tensions of 108 cm (sandy loam), 105 cm (silt loam), 94 cm (clay), and 56 cm (sand) with flow rates of 0.14 cm/day, 0.14 cm/day, 0.10 cm/day and 0.05 cm/day, respectively. For $R_b = 1,000$ days and $H_o = 5$ cm, a clay is more permeable than a sand.

2) Identical barriers induce different moisture tensions in different soils because their hydraulic effect is not only dependent on their own resistance but also on the capillary properties of the underlying porous medium. For example, a crust with $R_b = 100$ days and $H_o = 5$ cm induces tensions of 80 cm (sandy loam), 45 cm (silt loam), 40 cm (sand), and 20 cm (clay).

3) Increasing the hydraulic head on top of a barrier with fixed R_b increases the flow rate and reduces the tensions in the soil, but effects are generally minor. For example, a barrier induces flow rates of 0.5 cm/day ($H_o = 5$), 0.7 cm/day ($H_o = 30$) and 1 cm/day ($H_o = 60$) in sand. Corresponding tensions are 42, 40 and 38 cm, respectively. The effect of increasing the head is a function of the capillary properties of the porous medium and thus, the shape of the K curve.

4) Barriers with a small resistance ($R_b = 5$ days) will not affect the clay soil (except for $H_o = 5$ where a tension of 3 cm and a flow rate of 1.8 cm/day is induced). In fact, the R_b and H_o curves reflect hydraulic conditions imposed by the crust and the K curves reflect those allowed by the soils. The most limiting of the two (the K curve in the example discussed) determines conditions if the curves do not cross.

In summary, the higher the "crust" resistance or the lower the steady rate of application of water, the higher the soil moisture tension in the underlying soil and the lower the water content and the relevant hydraulic

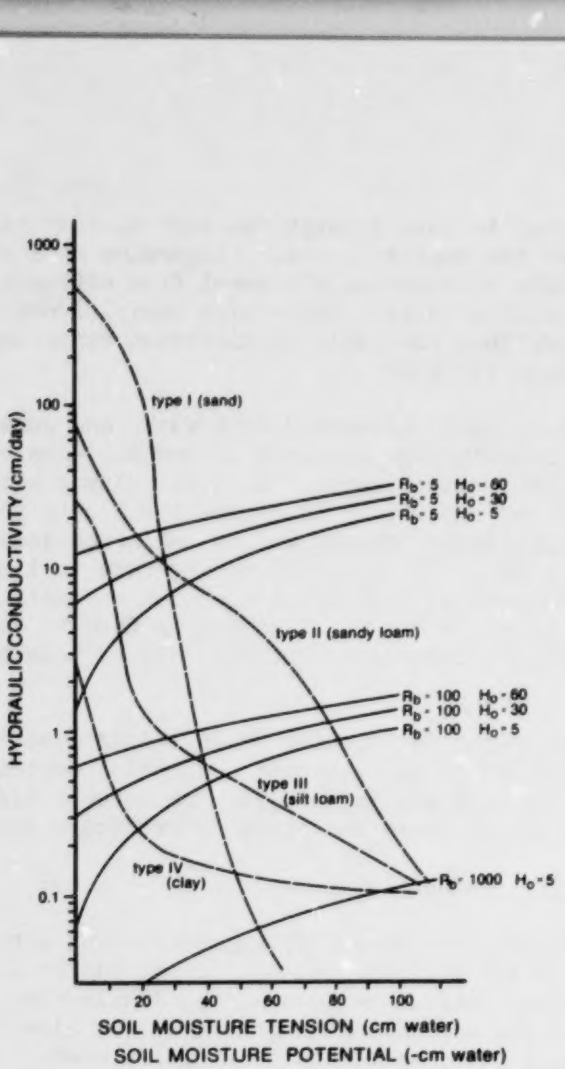


Figure B-9. Hydraulic conductivity curves for four major types of soil and curves expressing the hydraulic effects of impeding barriers of different resistances (see text) (Bouma, 1975).

conductivity (K). These characteristics apply to steady-state conditions in a one-dimensional system, where, at a hydraulic gradient of 1 cm/cm, flow rates are equal to the hydraulic conductivity. More complex flow systems, for example, those where the moisture content is changing with time, need more complex mathematical expressions which are beyond the scope of this discussion.

Flow dispersion--The longer a volume of liquid waste is kept in contact with soil particles, the greater the chance of good purification. If flow is such that all of the liquid present at equilibrium (one pore volume) will be "pushed out" before the applied liquid passes, then simple flow procedures can be used to explain the system. If some of the applied liquid passes through the soil before all of the liquid initially present leaves, then hydrodynamic dispersion has occurred. Physically, this can happen if channels or pores allow water to move through the soil so fast that the water does not have time to enter the smaller pores. Dispersion or short circuiting has been used to explain poor bacteria removal from effluent passing through soil columns at high loading rates. Dispersion coefficients (D) and breakthrough curves have been defined for soils of different structures and loading (Anderson and Bouma, 1977a,b).

For columns initially saturated with water and dosed with an unlimited amount of water, breakthrough occurred before 0.5 pore volumes had passed in subangular blocky structured soils. In soil columns with prismatic structure, breakthrough occurred after that in subangular blocky soils. If the structured soils were initially drained and an unlimited dose was applied, breakthrough was very rapid. Reducing the dosing rate to 1 cm/day delayed the breakthrough considerably; however, dispersion was still prominent. Addition of an artificial crust at the infiltrative surface to simulate a clogging layer of a soil absorption system further delayed breakthrough and reduced dispersion.

The high dispersion found using an unlimited dose in previously drained soil may be controlled in on-site soil absorption systems by controlling the loading rate or allowing limited clogging to occur. Differences in dispersion are related to soil structure and possibly predicted from soil descriptions.

Methods of Hydraulic Conductivity Measurement--

In this section, five permeability measurement methods are discussed. These include: 1) the field percolation test; 2) the use of small soil cores; 3) the instantaneous profile method; 4) the double-tube method; and 5) the crust test. Detailed experimental procedures are given for each method, except for the percolation test, which has been widely used. The advantages and shortcomings of the methods are discussed in some detail along with the method description.

The field percolation test--Henry Ryon is generally credited as the originator of the field percolation test in 1926 (Federick, 1948). He used it as a rough empirical measure of the ability of the soil to accept septic tank effluent over a long period of time. Since that time it has been widely used to evaluate site suitability for soil absorption field construction because it is simple and easy to use. However, the test has been criticized because it is generally an ineffective predictor of absorption field performance.

Procedure--There are many variations on the percolation test. The following description summarizes the technique used by the Wisconsin State Division of Health (1967).

A hole is augered to the soil horizon in which a proposed drainage field is to be constructed. A cylindrical hole with a diameter ranging from 10 cm (4 inches) to 25 cm (10 inches) and often larger is used. The bottom and sides of this hole are carefully scratched to ensure an unsmeared surface and 5 cm of gravel or coarse sand is placed in the hole. Water is carefully poured into the hole so that the soil is not disturbed by violent water flow.

Usually water is introduced to a depth of 30 cm. If the 30 cm of water flows away rapidly (less than 10 min), indicating high permeability the test can proceed at once. If more than 10 minutes are required for seepage of the water, then water must be maintained at 30 cm depth for 4 hours and a period of 16 to 30 hours must be allowed for swelling of the soil before measurements can be made.

Measurements are made of the amount of head change that occurs in thirty minutes. The water depth is 15 cm at the beginning of each experiment. When these measurements remain nearly constant, the percolation rate is calculated and expressed in minutes per inch or inches per hour.

Comments--Many papers have addressed problems that are associated with the percolation test (Winneberger and McGauhey, 1965; Hill, 1966; Bouma, 1971; Luce, 1973; Chan, 1976). The test itself has several weaknesses. The flow of water from a hole can be used as a rough means of ranking the soils ability to conduct water. But to extrapolate the rate of flow under these conditions to the design of a subsurface drainfield very often leads to inadequate sizing of the system. The reasons for this failure are easily recognized. The percolation test measures flow from a hole out in all directions. The flow rate or "perc rate" measured is based only on the dimensions of that hole and cannot readily be used to calculite the effective flow rate of a given seepage field trench. The percolation rate is a combination of the vertical and horizontal component of soil hydraulic conductivity.

The diameter of the hole used for the test does not appear to influence the resulting measurements in practice (Chan, 1976). Logically, a change in rate would be expected, but any differences seem to be cloaked by the large general variation within the technique. These variations may also be due in part to soil and climatic conditions.

The depth to which water stands in the hole does make a notable difference in measured percolation rate. Increasing the height of water in the test hole can more than double the percolation rate (Chan, 1976). There is less variation when a constant head of 12 cm is maintained than when a falling head is used as described here. For a series of falling head measurements, Bouma (1971) found the coefficient of variation to be 54 percent, as opposed to 35 percent with the constant head procedure in the same soil.

Several other factors affect the test. The initial moisture condition of the soil can lead to errors. Percolation rates are often higher in summer because a longer wetting time is required in dry soils, particularly those with fine texture. Slaking of the bottom and sidewalls of the hole can occur, restricting flow. Entrapment of air in soil pores can prevent reproducible saturated conditions from occurring. There is not space to discuss these further, but a good discussion of the percolation test is presented by Chan (1976).

The great failing of the percolation test, as with all saturated permeability tests, is that it cannot predict the rate of flow from a drainage field after a clogging layer has been formed. To do this, the unsaturated hydraulic conductivity and the soil moisture potential under the drain field must be known.

The use of small soil cores for laboratory measurement of hydraulic conductivity--The measurement of saturated hydraulic conductivity through packed sand and soil columns has been a useful tool for hydrologists since the early experiments of Darcy (1856). Since that time the method has been applied to measurement of saturated flow in undisturbed soil cores which maintain much of the natural soil structure. Richards (1949) extended this method further to measure hydraulic conductivity of soil for unsaturated conditions. Measurements of K from soil cores and columns generally require the establishment of a steady state flow of water through the soil. Although many variations on this method have been used, the basic procedure remains the same.

The use of soil cores has several logistical advantages. Cores can be collected rather easily and quickly in the field. The cores can be run in the laboratory under controlled conditions. Although the test conditions are reproducible, the results may be highly variable. This method yields information on saturated and unsaturated hydraulic conductivity.

Experimental apparatus--Special laboratory apparatus is required for the measurement of unsaturated K by this method. Figure B-10 (Richards, 1949), represents an arrangement that allows the use of 7.5 cm diameter soil cores of up to 10 cm height. The exact construction of this chamber can have several forms depending on the size and shape of samples used. Seals can be made by the use of "O" rings to fit the cylindrical soil core (Klute, 1965). The sample core is enclosed in a chamber and two tensiometers T and T are inserted into the soil from the side. These will be used to calculate the potential gradient across the sample. Porous plates are placed into firm contact with the soil samples upper and lower surface and sealed to the outer wall of the chamber. Flow into the chamber (Q_1) is measured in a burette that has a constant head device (M), a Mariotte tube inserted. Outflow (Q_2) is via a drip point connecting to the chamber below the second porous plate. This liquid is collected in a graduated container. The distance that the drip point is below the soil sample controls the applied moisture potential by adjusting the heights of the inflow water container and the drip point. The applied hydraulic gradient can be varied. This arrangement is accurate in the soil moisture potential range of 0 to -200 cm water, and cannot be used for potentials less than -750 cm, because of the vaporization of water.

Saturated hydraulic conductivity (K_{sat}) can be measured by a simpler technique requiring only the constant application of water to the sample's upper surface under a minimum hydraulic head. Water flows out of the core and this flux is taken as K at saturation. This technique is very dependent on the size of the soil sample used. Anderson and Bouma (1973) have shown that the value and the amount of variation of the measured K is inversely proportional to the length of the soil core used. This suggests that the values for K recorded may be arbitrary numbers of limited use. Standardized core sizes must be used for this purpose.

Procedure--The hydraulic conductivity of the soil sample at a given moisture potential ψ_m is

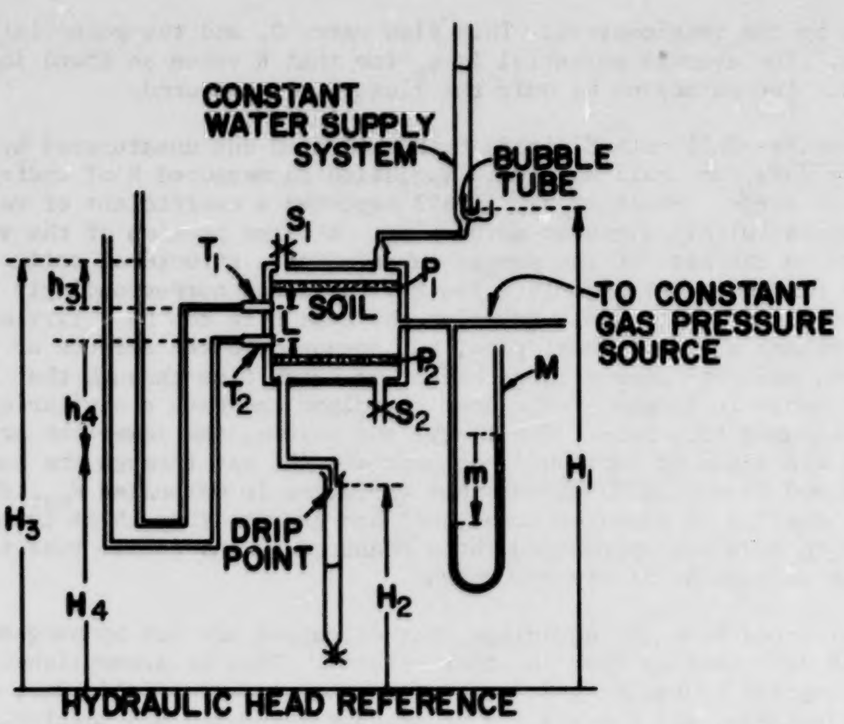

CONSTANT
WATER SUPPLY
SYSTEM
BUBBLE
TUBE
S_1
T_1
h_3
P_1
SOIL
L
TO CONSTANT
GAS PRESSURE
SOURCE
T_2
P_2
M
S_2
h_4
m
H_1
H_3
DRIP
POINT
H_4
H_2
HYDRAULIC HEAD REFERENCE

Figure B-10. Diagram of apparatus for steady-state method of measurement of conductivity of unsaturated soil. The meaning of the symbols is explained in the text (Klute, 1965).

$$K(\psi_m) = \frac{Q}{A} \cdot \frac{L}{\Delta H}$$

for downward vertical flow, where

Q = net flow rate (cm^3/day),

A = surface area of the soil sample (cm^2),

$K(\psi_m)$ = hydraulic conductivity for moisture potential ψ (cm/day),

ΔH = the difference in hydraulic head (cm),

L = the length of the core (cm).

For the special case of gravity drainage, the last term ($L/\Delta H$) is nearly equal to one and can be dropped from the equation.

The procedure is carried out by securing the sample in the chamber with porous plates positioned and tensiometers inserted. Water is applied to the upper plate until steady state flow is achieved and constant potentials are recorded by the tensiometers. This flow rate, Q, and the potentials are recorded. The average potential is ψ_m for that K value as found in the above equation. For saturated K, only the flux, Q, is measured.

Comments--This method yields both saturated and unsaturated hydraulic conductivity data for small samples. Variation in measured K of undisturbed soil samples is large. Mason et al. (1957) reported a coefficient of variation of 70 percent within sites at saturation. A large portion of the variation is dependent on the size of the sample taken. For a structured soil, a 7.5 cm diameter ring may contain only a few soil peds and correspondingly few structural pores. Another sample taken at the same site can be a little different, having perhaps one additional pore, but compared to the effects of the other few pores, this may have a large effect on water flow through that sample. Also, a change in length of the core or column can have a similar effect on the measured flow rate. The longer the column, the lower the probability that a given chain of pores will connect all the way through the sample. Anderson and Bouma (1973) showed that variation in saturated K_{sat} decreased markedly for 7.5 cm diameter cores that are greater than 15 cm in length. At this length K values approached those measured by the double tube test for a soil with subangular blocky structure.

Soil cores have the advantage that entrapped air can be purged from the pores more readily than in other methods. This is accomplished by the use of reversed hydraulic gradient. Also, the reduced field time, fewer weather problems, and the ability to measure saturated and unsaturated K are advantages. The inherent disadvantages are that significant laboratory time is required, variation in results is large, and relatively large cores or large numbers of cores are required to reduce this variation.

The instantaneous profile method for measuring hydraulic conductivity of unsaturated soil in situ--In the instantaneous-profile method, a fallow plot of soil is artificially wetted to saturation. The plot is then covered to prevent evapotranspiration, and internal drainage occurs. By the use of tensiometers, placed at depths corresponding to the lower part of major horizons, soil moisture potentials are measured. Moisture contents at these depths are also determined simultaneously. These two characteristics are measured frequently, starting at saturation, which represents time zero.

From these data, the hydraulic gradient ($\partial H/\partial Z$) and the change in moisture content over time ($\partial\theta/\partial t$) can be obtained and subsequently the hydraulic conductivity can be calculated.

Theory--Taking the general equation of vertical hydraulic flow in a soil profile

$$\partial\theta/\partial t = \frac{\partial}{\partial Z} K(\theta) \frac{\partial H}{\partial Z}$$

where θ = volumetric moisture content,

H = hydraulic head ($H = \psi + Z$),

$K(\theta)$ = hydraulic conductivity at moisture content θ,

Z = depth (positive downward),

t = time.

By integration and rearrangement, we find

$$\int_0^Z \frac{\partial\theta}{\partial t}\, dZ = K(\theta) \cdot \frac{\partial H}{\partial Z} \quad \text{or} \quad K(\theta) = \frac{\partial\theta/\partial t}{\partial H/\partial Z} \cdot Z$$

In a multilayered profile the factor $(\partial\theta/\partial t) \cdot Z$ becomes the sum of

$$(\frac{\partial\theta}{\partial t})_1\, Z_1 + (\frac{\partial\theta}{\partial t})_2 + (Z_2 - Z_1) + (\frac{\partial\theta}{\partial t})_3 \cdot (Z_3 - Z_2), \text{ etc...}$$

where Z_1, Z_2, Z_3 are the depths of the lower boundaries of the respective soil horizons 1, 2, 3, in order of depth and the quantities involving Z are respective horizon thicknesses. The individual $(\partial\theta/\partial t)$ factors correspond to the respective horizons, as indicated. The factor $(\partial H/\partial Z)$ can be derived and $K(\theta)$ calculated.

Soil moisture content can be plotted versus time (zero time is when the profile is saturated and drainage begins) and the slope of the resulting curve gives $(\partial\theta/\partial t)$ for any particular time. The curve is usually in the form of a hyperbolic function, as shown in Figure B-11.

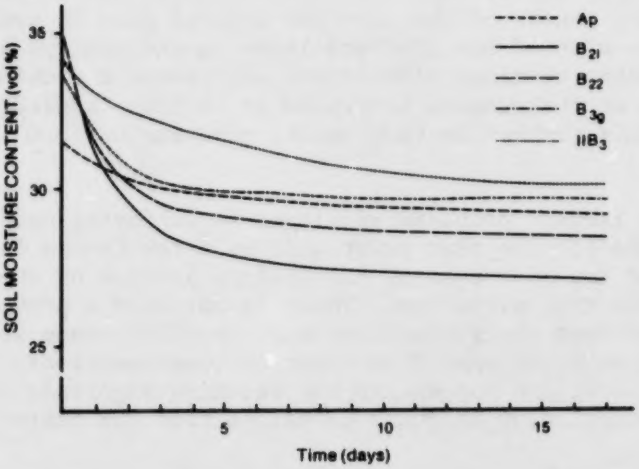


Figure B-11. Moisture content as a function of time during drainage of an initially saturated profile (used for calculations of the instantaneous-profile method).

The hydraulic gradient ($\partial H/\partial Z$) of an internally draining, initially wetted profile is often unity. The exact value is the slope of the line created by plotting hydraulic head ($H = \psi + Z$) versus the depth A to the tensiometer cup where the head is measured (Fig. B-12). This slope varies with time, and must therefore be determined for each time of observation at which K is calculated. This will be discussed in greater detail in a specific example later.

Site selection and preparation--A nearly level site is selected for the experiment so that water may be ponded more or less uniformly over the entire area. To retain the water, a simple earthen dike can be constructed surrounding the plot. A water depth of about 2 or 3 cm over all portions of the pond is desired, but may be difficult to achieve. On slopes of low grade, uniform distribution of water is not a problem, but on steeper slopes, uneven infiltration and subsurface flow can occur. Figure B-13(A) illustrates the desired flow pattern in the experimental plot. Water infiltrating over the entire area of the plot moves generally downward under the influence of gravity. Because the soil surrounding the plot is drier than the soil directly under the pond, lateral movement of water is expected to occur. This drier surrounding soil exerts a tension on the water inside the experimental volume, pulling it away from the center of the plot. This effect is greatest at the boundaries of the area and decreases toward the center. As the flow lines indicate, flow is nearly vertical below the center of the plot. Here, approximate one-dimensional flow is achieved. All the details of plot preparation are designed to maintain vertical drainage at this point, isolating this zone from the effects of lateral flow and from changes of soil moisture content due to external factors such as evapotranspiration and precipitation.

Figure B-13(B) illustrates the effect of slope on water movement for a plot where uniform infiltration occurs. Vertical flow is seen to be shifted somewhat from the center of the plot and lateral flow is seen to be greatest on the downslope side of the plot and least on the upslope side. In such a case, knowing where vertical flow occurs may become a problem. The location and orientation of instruments is dependent on this knowledge. On gentle slopes ($< 3\%$) this problem is very small, assuming uniform infiltration over the entire area.

One of the largest problems on slopes is achieving uniform infiltration. Often the surface is such that water will be a few inches deeper on the lower portion of the plot than on the upslope portion of the area. Figure B-13(C) indicates this situation. There is not only a problem with distributing the water over the surface but also the difference in hydraulic head will cause flow rates to vary from upper to lower sections. The length of the flow lines in the figure represents the relative magnitude of flow rates. Notice that vertical flow is found downslope from the center of the experimental area.

It can be seen from these examples that the ideal case will rarely be encountered in the field experiment. A sloping site will cause nonvertical flow under the center of the plot and uniform distribution of water will become difficult to achieve.

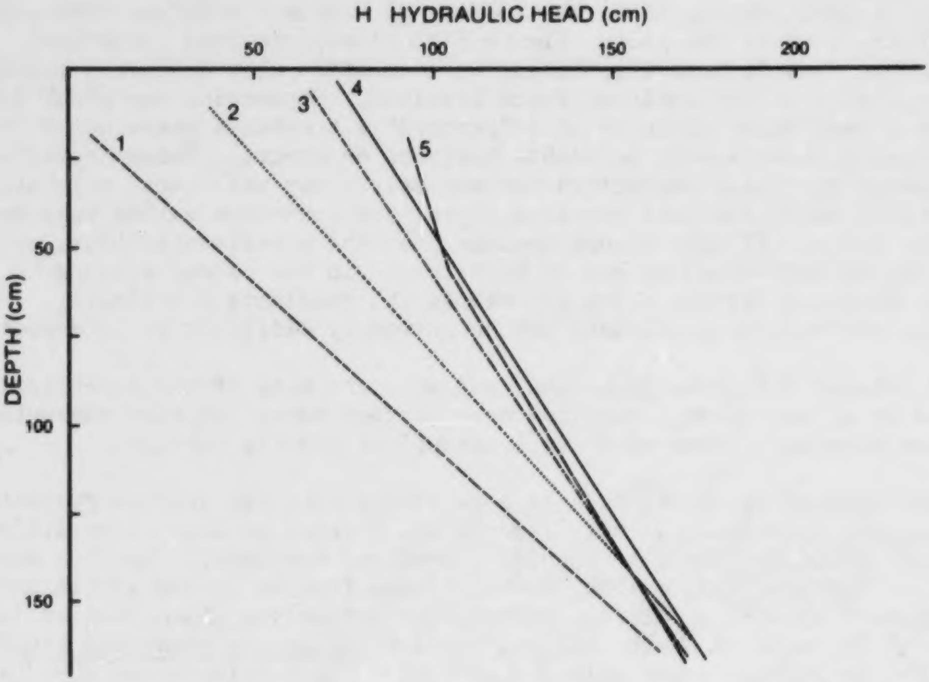


Figure B-12. Total potential as a function of depth and time during drainage of an initially saturated profile.

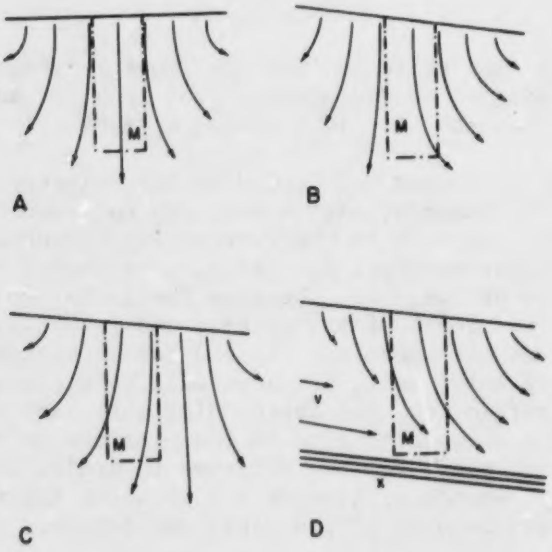


Figure B-13. Diagrams illustrating possible flow patterns occurring when saturating soil for the instantaneous-profile method.

Other problems may arise due to flow of external moisture from upslope through the soil of the plot. Figure B-13(D) depicts this situation. This directional flow is most significant at saturation when downslope movement due to gravity is the dominant force involved. Saturation may occur in the form of a real water table or of a "perched" water table where water is temporarily ponded above slowly permeable horizons or strata. Water contained in this manner by a less conductive horizon below, may well cause saturation in the horizon above the less pervious layer, and downslope rather than vertical flow may occur. If this occurs upslope from the experimental plot, external water may be introduced at one or more levels in the pedon, setting up a complex drainage pattern which may change the gradients drastically. Data obtained under these conditions can be extremely difficult to interpret.

A related situation is encountered when the site of the experiment is located in a depression, where not only surface water but also subsurface moisture flow is concentrated due to downslope gravity movement.

In homogeneous media, such as some sands, a nearly uniform gradient is found during drainage. In multilayered soils where horizons have different hydraulic conductivities, a nonuniform gradient develops. Impeding horizons and other features will prevent free drainage from occurring within the experimental volume of soil. Soil moisture potentials above such an impeding layer may approach or reach zero (saturation) while the potential beneath the layer can be higher (lower moisture content). Such a layer can then behave as an impeding crust, preventing saturation from occurring in horizons below itself. Therefore, a part of a given multilayered soil may not reach saturation, and K values close to saturation for the unsaturated horizons cannot be determined by a free-drainage method such as this. Examples of some soil features that can act as impeding layers are a plow sole, an argillic horizon, and pans.

Once a site has been selected, the area must be prepared for the experiment. These efforts are designed to isolate the plot from its surrounding environment and to ensure that the plot is a closed system.

The size of the plot used may depend on the moisture condition of the surrounding soil. For example, when a very dry soil surrounds the plot, lateral movement from the plot to the surrounding environment can be great. This may lead to exaggerated drainage rates, as recorded by the instruments located at the center of the plot. Because the latter movement of moisture is not uniform over the depth of the profile and is usually greatest for the uppermost horizon, the head gradient ($\partial H/\partial Z$) can be exaggerated. If the plot is surrounded by very moist soil, drainage will take place very slowly. Moisture may enter the system from the surrounding soil, and the length of time required to achieve a certain tension in the plot may be very long. This may be because there is no place for the moisture to drain, a moisture regime that rarely achieves a low enough tension or a high water table which interferes with the head gradient because of its capillary fringe.

A circular plot of a diameter of 3 meters has been used successfully (Davidson et al., 1971), but under extreme conditions of drying in the surrounding soil, this may be an inadequate volume to allow undisturbed internal drainage at the center of the site (Vepraskas et al., 1974). In such a case, the diameter of the plot may be expanded to ensure unaffected drainage at the center. On the other hand, in some environments a smaller plot may be sufficient to eliminate boundary effects over a low-tension range.

The site is selected and the necessary area is estimated. Vegetation is either entirely or partially removed. Cutting vegetation off at ground level is more effective than pulling or hoeing it, as this will not disturb the infiltration surface. Plant removal is relatively simple in open fields but on wooded sites where shrub and tree roots permeate the soil, little can be done without destroying these plants. The root systems of trees on a hot summer day can noticeably affect the moisture content of a profile, thus interfering with the experiment. Selection of a site at a location farther from trees may bias the experiment somewhat, but the results will be more representative of the actual conductivities of the soil at that point. Also, if the roots of the plants remain, revegetation is faster and the soil is less subject to erosion.

Walking on the plot should be held to a minimum when the plot is dry and should be eliminated entirely if the soil is moist or wet. This precaution is necessary to prevent puddling of the soil surface. Placing a sheet of plywood on the plot or extending planks across it supported with blocks at the boundary are two simple methods to keep traffic off the soil.

An earthen dike about 7 cm high can be constructed easily with sod from outside the boundaries. The purpose of this dike is to allow ponding of water over the area of the plot. It also prevents any surface runoff from upslope from flowing over the plot, rewetting it, and interfering with the experiment. Figure B-14 shows a prepared plot with the earthen dike in place.

After the profile has been wetted, it must be isolated from such factors as precipitation and evapotranspiration. These can be controlled with two layers of plastic sheeting large enough to cover the plot. One should be sufficiently broad to allow it to be supported at the center with the sides sloping out beyond the boundary of the plot. The smaller of the sheets should be spread over the freshly wetted area. Cut a hole about one foot in diameter in the center of it so that the instruments can protrude for convenient maintenance and recording. The earth between the instruments can then be covered over with aluminum foil or small plastic sheets. If the plastic sheet is nontransparent preferably black, sunlight will not easily penetrate to the soil surface and plant growth will be severely retarded. With a transparent sheet, plants will grow quickly allowing transpiration to occur with removal of moisture from the soil. The sheets are held down at the edges with weights or stakes. The larger plastic sheet is used as a tent to shed precipitation. It is supported at the center of the plot by a post about 1-meter high, on the top of which is a circular wooden disk about 25 cm in diameter. The disk should not have rough edges that would tear the plastic. The circular design distributes the weight of the plastic uniformly and prevents punctures. Figure B-15 shows a prepared plot with the second plastic sheet in position.

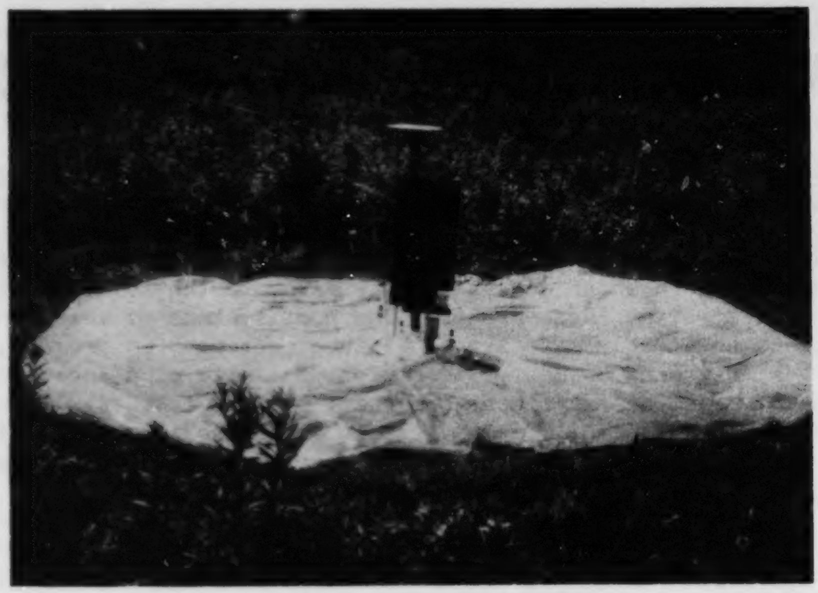


Figure B-14. Picture of prepared plot for the instantaneous-profile method with the first plastic sheet in position (Bouma et al., 1974a).

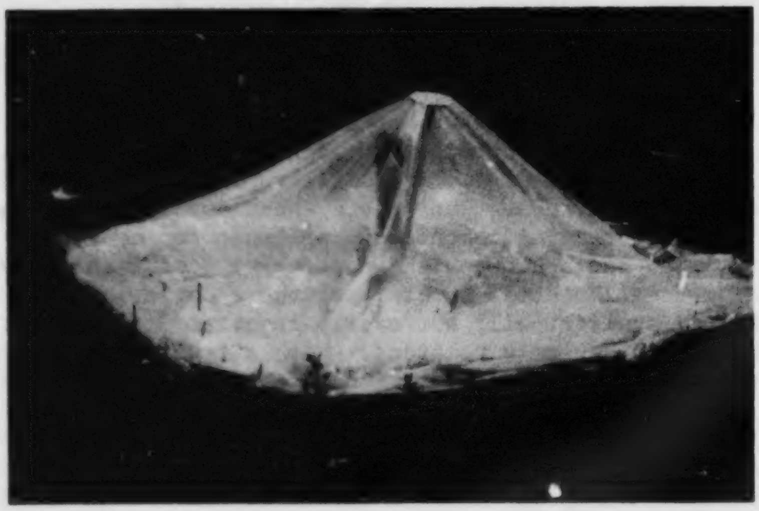


Figure B-15. Picture of prepared plots with the second plastic sheet (b) in position (Bouma et al., 1974a).

Alternate forms of plot preparation are possible. One involves the digging of a ringlike trench around a 1-meter diameter undisturbed column of soil (Anderson and Bouma, 1973). The trench is made to a depth somewhat below the deepest horizon for which the conductivity is to be determined. Following this, a detailed profile description can be made on the wall of the column, providing accurate horizon boundaries for location of the tensiometers. The sides of the column must be covered with plastic sheeting or aluminum foil to prevent evaporation. The surface is prepared as described previously, and a ring or dike is constructed to retain the ponded water on the top for wetting. Figure B-16 shows a prepared column arrangement.

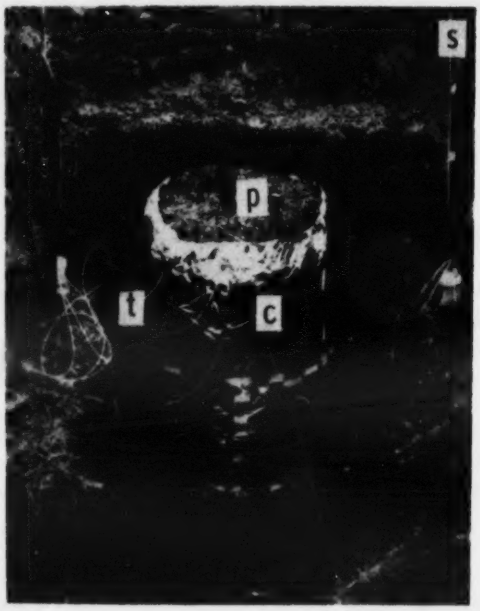


Figure B-16. Large excavated column for running the instantaneous-profile method on sites where the regular procedure cannot be applied. p = metal pipe for neutron probe; c = soil column; t = tensiometers and s = tensiometer boards with calibrated scales for reading moisture tensions (Bouma et al., 1974a).

This arrangement has the distinct advantage that tensiometers can be implanted from the sides and the problems of vertical placement can be eliminated. For this application, small 0.6-cm diameter tensiometers, as described for the crust test method, can be used. The neutron moisture probe is a convenient tool for moisture-content measurement here.

One of the main advantages of the column method is that one-dimensional flow is maintained. There is no lateral interference with drainage or downslope moisture flow. Internal drainage proceeds uninterrupted. Effects of neighboring vegetation are eliminated and problems associated with slope are greatly reduced. However, the procedure is elaborate and costly.

Instrumentation--Accurate soil-moisture-tension and soil-moisture-content measurements are necessary for the implementation of this technique. The careful placement of tensiometers in specific horizons of the profile and the sealing of these tensiometers in place will be discussed here, followed by a discussion of methods of moisture-content determination. Before this, it is important to point out that an accurate profile description of the soil at or adjacent to the site is essential for the determination of depths of placement of tensiometers and for the useful interpretation of results. This technique can be applied very well to major horizons, but may not be specific enough for smaller soil features, such as some subhorizons.

Normally, the depth of tensiometer placement is meant to coincide with the lower boundary of the horizon to be studied. Placement a few centimeters above this boundary is perhaps more realistic since the tensiometer does not read pinpoint tensions, but measures the tension of the small volume surrounding the cup. It is the difference in tension across the layer which is needed to find the conductivity of that layer. This is the head gradient. So if only one horizon is to be studied, a tensiometer is needed at both the top and bottom of that particular horizon to define that gradient (see Fig. B-17).

Tensiometry--Tensiometers with porous cup measuring 1.9-cm OD and 5-cm long, attached to clear plastic tubing of 2.0-cm OD, and cut to various lengths have been used. These were placed in the soil by the following methods.

Using a screw auger, a hole is bored of slightly larger diameter than the tensiometer itself. This hole is made 5 cm less than the depth of desired tension measurement. A push auger of a smaller diameter than that of the tensiometer may be used to extend the depth of the hole, or if the soil is moist enough, the tensiometer may be pushed into final position. This last technique will disrupt some structure and may cause rather severe puddling surrounding the cup.

A cavity along the side of the tensiometer may conduct water to the vicinity of the cup leading to tension measurements not representative of that horizon. For this reason, the space surrounding the tensiometer must be filled or sealed.

To seal the auger hole around the tensiometer, a liquid slurry is poured into the empty hole in such a way as not to trap air at some point along the cavity. This is accomplished by pouring the slurry just to one side of the hole so that it will flow as a stream down one wall of the hole thus not preventing the escape of air from below. Once the hole is filled, the tensiometer is pushed into the slurry forcing it out at the surface to form a cap over the hole (see Fig. B-18). If some air has been trapped, it will also rise to the surface. The displacement causes the slurry to be forced into

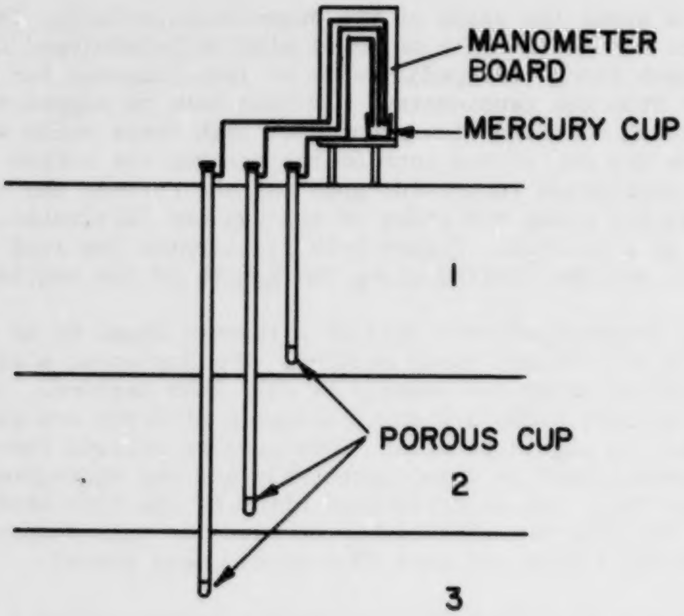

MANOMETER BOARD
MERCURY CUP
1
POROUS CUP
2
3

Figure B-17. Schematic diagram showing appropriate locations of tensiometers in a soil profile with three horizons (Bouma et al., 1974a).

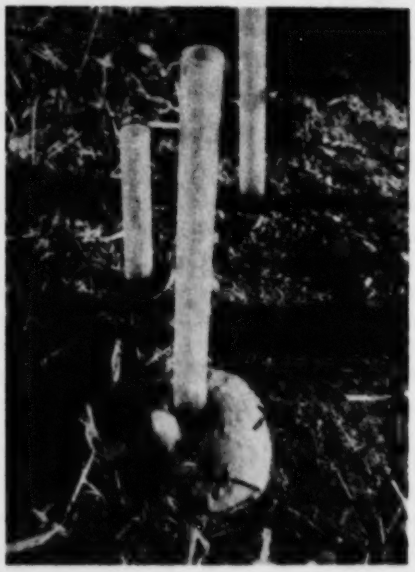


Figure B-18. Installed vertical tensiometer with slurry forced out at the surface to form a cap over the hole (Bouma et al., 1974a).

holes and cracks along the walls of the auger hole, reducing the possibility of lateral water movement. In a dry soil with well-developed structure, slurry may be forced back along interpedal voids or into biopores for a distance of a few centimeters from the tensiometer. If this were to happen to any great extent, the natural moisture flow associated with these voids would be affected. Where the cup of the tensiometer reaches the bottom of the hole, it must be forced into place to provide good contact between the cup and the soil. Some smearing along the sides of the cup are inevitable, but smearing should be held to a minimum. Figure B-19 illustrates the good contact between the cup and soil and the sealing along the length of the tensiometer.

The slurry is prepared from soil of a texture equal to or slightly heavier than the texture of horizons being studied. For instance, a silty clay loam is used for profiles which are largely of silt loam texture. This material is placed in a shallow basin and small aliquots of water are added, with mixing, until a viscous liquid results. Time must be allowed for the swelling of clays. The material must be mixed thoroughly and any aggregates or clods should be broken down. An electric hand mixer of the type used for mixing cake batter in the home may effectively be used for this purpose. The slurry should be a viscous liquid and must flow evenly when poured.

Another method of tensiometer placement requires making a hole to the required depth with a push auger of slightly smaller diameter than that of the tensiometer itself. The tensiometer is then forced into place, assuring good contact with the soil.

Either of these techniques provide good contact between the porcelain cup and the soil. However, the first method assumes that there is a cavity along the tensiometer and procedures are defined to fill it, while the second method relies on the tensiometer being forced in for the length of the hole to a close fit. It is important that there be no unnatural vertical cavities in the profile, since at saturation such a cavity will act as a shortcircuit and conduct water rapidly downward, creating an unnatural moisture distribution.

Small holes are bored in the walls of the plastic tube for insertion of a 0.3-cm flexible tube that connects the plastic tensiometer tube to the mercury cup via a manometer board as shown in Figure B-20. The scales for the boards are graduated to read cm of water and can be purchased individually. An adequately large mercury reservoir is selected so that the surface of the mercury in the reservoir does not fluctuate greatly as tensions vary. A top for the mercury reservoir is recommended with holes drilled in the cap to accommodate the 0.3-cm tubes. The reservoir should be taped or otherwise anchored to the manometer board to avoid spillage.

The tensiometers are filled with de-aired water by pouring the water down the inside of one wall of the plastic tube of the tensiometer until the shaft is completely full. The water is poured gently to avoid dissolving too much air in the water. Next, a 50 or 60 ml plastic syringe with a single-hole rubber stopper (No. 1) fixed on the tip is inserted into the plastic hole (Fig. B-21) with the stopper fitting tightly in the plastic tube. By pushing the plunger of the syringe forward, water is forced into the system and fills

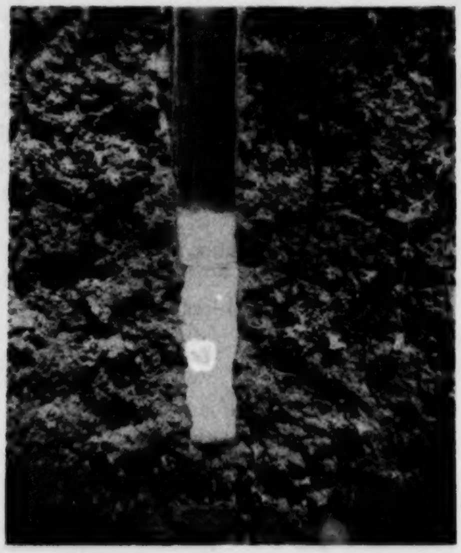


Figure B-19. Excavated tensiometer cup showing good contact between soil and porous cup and complete filling of the hole above the cup with slurry (Bouma et al., 1974a).

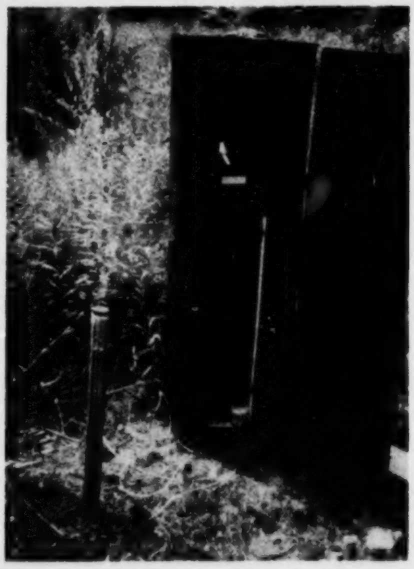


Figure B-20. Tensiometer assemblage, showing the connection of the plastic tube (T) with the manometer board (B) and mercury cup (M) through 0.3 cm flexible tubing (t). The black box with a lock serves as a deterrant to vandalism. The access tube (A) for the neutron probe is next to the tensiometers (Bouma et al., 1974a).

the 0.3-cm plastic tubing. When water flows out of the top of the mercury reservoir the syringe is removed, the tube filled to overflowing and a solid rubber stopper (No. 1) is inserted to close the system. With the tensiometers shaft-oriented vertically, any air in the water tends to rise to the top of the shaft under the rubber stopper. Even though de-aired water is used, some air may appear here, particularly at higher tensions. Whenever air accumulates under the stopper it should be removed by filling the tube with more water. Gas in the system can lead to erroneous tension measurement, due to expansion and contraction with temperature variations. If a lot of air is present or if the condition persists for several days, this may indicate a leak in the system, usually a pinhole or crack in the 0.3-cm tubing or a bad seal of the fine tubing to the tensiometer shaft.

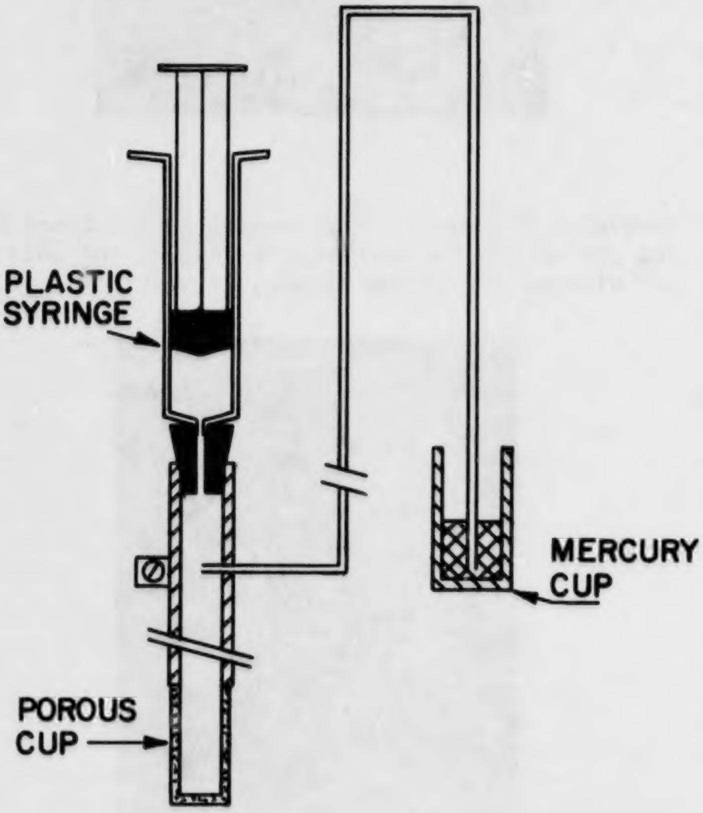


Figure B-21. Filling of the 0.3 cm flexible tubing, using a plastic syringe which connects the water-filled plastic tube and porous cup and the mercury cup (Bouma et al., 1974a).

Soil moisture content--Two methods can be used here: 1) indirect values derived from moisture retention curves and 2) direct values derived from in situ neutron probe measurements.

Soil moisture content can be determined by the use of moisture retention (desorption) data for the given horizons (Davidson et al., 1971). Soil cores (7.5 cm diameter, 5 cm high) are taken adjacent to the plot at the depths to which tensiometers are to be placed. The plot is wetted and drainage occurs. In this case, only matric tension is recorded as the experiment progresses. The moisture content of a specific horizon is determined from the moisture retention curve by finding θ for the core at the tension recorded in the horizon at that time. The moisture content data are indirectly gained but can be used in the calculation of K. The limitation of this technique is that the soil core may not represent the horizon as a whole. Several cores may be needed for a good average of values. Double tensiometry may also be needed to ensure accuracy.

The neutron probe may also be used for this purpose. The neutron moisture probe consists of a fast neutron source and slow neutron detector. Fast neutrons travel radially into the surrounding soil where, if they strike the hydrogen atom of a water molecule, they are slowed considerably. The main source of hydrogen in the soil is water, and so, aside from the small background noise of the soil material itself, the number of slow neutrons detected is roughly proportional to the amount of water in the soil (Cannell and Asbell, 1974). A counter attached to the probe converts this information into digital form. With calibration, counts/minute can be translated to percent water by volume.

An access tube, 4.1 cm OD thin-walled metal conduit, is placed in a hole augered several inches deeper than the deepest horizon at which measurement is desired. The pipe should fit the hole tightly and may need to be driven into place. The hole should not be filled in around the pipe as this may lead to unrepresentative moisture contents. The probe can then be lowered in the access tube to the depth where measurements are required, measured from the soil surface to the neutron source. Note that the size of the roughly spherical volume of soil for which the moisture content is being determined varies with the moisture content of the soil. When the soil is very wet, a small volume is required to slow the neutrons and, when the soil is very dry, a larger volume may be needed to slow the neutrons. This means that in drier soils, a larger sample volume is needed and the moisture probe becomes less specific for a single thin subhorizon. So the neutron probe cannot yield the specific information in a dry soil that it can in a wetter soil. If a measurement is taken near a horizon boundary, the resulting moisture content may be somewhere between that of each horizon.

Gravimetric determinations of the soil moisture content can be used. A limitation of the technique is that a large number of samples would need to be taken during the course of the experiment, and variability across the plot may lead to slight variations of the data. The change in water content over time will be recognized as $\partial\theta/\partial t$.

Experimental procedure--Now that the plot has been prepared and the instruments are in place, the plot can be wetted. Often it will be necessary to wet the plot a day or two before the start of the actual measurements. This allows time for the swelling of clays and adsorption of water by the peds. Wetting can be accomplished by running a hose from a nearby house or by the use of trailer-mounted, large-capacity water tanks. Impact of the water on the plot surface should be dissipated to avoid disturbing the surface and suspending soil material which in turn may cause clogging of some pores. Water is ponded on the surface to assure uniform infiltration. Application of water continues until instrument readings remain constant. At this point the wetting process ends, and as soon as water is no longer visible on the surface, the first measurements are taken (time = zero). The plot is then covered with plastic sheeting as previously described. Subsequent measurements are taken every hour or two for the first several hours. The time intervals may be extended as the rates of change decrease, until at about a week's time, measurements are taken every second day. The frequency of measurement depends on the rate of change of the gradients.

In some cases, a tensiometer may show positive tensions. If these are of low magnitude they may be due to temporary ponding of water on a less permeable feature or horizon. In such a case, a detailed soil profile description can be useful in determining the cause for this behavior. Another possible cause of positive tensions is a large cavity adjacent to the cup of the tensiometer. This may be due either to some large natural void, i.e., worm burrow or root channel, near the cup or an inadequate seal. Use of duplicate tensiometers decreases the probability of such an occurrence.

If a tensiometer does yield sizable positive matric tensions that do not decrease rapidly during the first few hours, the wetting procedure may need to be repeated (after resealing that tensiometer). If the questionable tensiometer shows a decreasing tension approaching the expected value, then in many cases the tensions of the first few hours can be approximated by extrapolation back to time zero (but not necessarily to zero tension). This procedure, although less reliable, can save time and eliminate the need to repeat the whole process.

Example calculation--After data have been collected over a period of time, covering the range of soil moisture tensions desired, hydraulic conductivities can be calculated. An example calculation is carried out here based on data gathered for a Batavia silt loam (Mollic Hapludalf) (Bouma et al., 1974a). Tensiometers were placed at depths of 30 cm, 55 cm, 81 cm, and 100 cm corresponding to the Ap, B1, B22, and B3 horizons, respectively.

Table 1 presents the matric tensions recorded at each depth for several times. At time 0 all tensiometers show positive tensions, indicating ponding of water above the tensiometer cups. Within 0.15 days these heads had been eliminated, except in the very heavy textured horizons.

Table 1 also presents soil moisture content (θ) data for the same horizon depths determined at the times that tensions were recorded. These numbers were obtained by conversion of neutron probe counts by use of a standard curve for the probe. These moisture contents (expressed as %vol. are plotted

TABLE B-1. SOIL MOISTURE PROBE DATA

		ψ_m (matric tension, cm)					θ (Vol %)				
Time	Horizon	Ap	B1	B22	B3	IIB3	Ap	B1	B22	B3	IIB3
(days)	depth(cm)	30	55	81	110	160	30	55	81	110	160
0.0		--	+36	+29	+25	+11	29	30	31	30	28
0.15		- 9	- 7	0	+15	+ 2					
3.0		-36	-25	-11	+15	+ 6	29	28	27	29	29
6.75		-44	-31	-19	-15	- 3	28	29	28	28	30
10.0							26	26	25	26	28
12.0		-50	-33	-18	-15	- 7	28	27	26	28	28
17.0							28	26	26	28	28
18.0		-57	-38	-21	-18	- 6					

as a function of time of measurement in Figure B-11. A curve is plotted in this way for each of the horizons studied. From this graph the slope of each curve at the times of measurement are derived. These slopes are the gradient $\partial\theta/\partial t$ ($cm^3/cm^3/t$) and appear in Table 2.

Hydraulic head is the recorded matric tension at a particular depth and time, plus the gravity head affecting that depth. The gravity head is the depth to the middle of the tensiometer cup down from the ground surface. Figure B-12 is the result of plotting hydraulic head (H) versus the depth (Z). For any given time the set of readings will form an approximate line or series of lines whose slope varies as drainage proceeds. This slope represents the desired gradient $\partial H/\partial Z$. The gradient can be found for specific times after the beginning of the experiment and is recorded in Table 3 for the respective times for each depth. For the cases where a single line is not accurate, two or three slopes may need to be taken, each one specific for the horizon at which it occurs.

Following the example of Hillel et al. (1972), incremental flux is calculated for each horizon and time (Table 3) by multiplying the moisture content variation by the thickness (dZ) of the horizon affected. The sum of these increments is the flux q for the depth Z of the deepest horizon. K is calculated by dividing q by $\partial H/\partial Z$ (Table 3) for each time. Also listed in this table are the corresponding moisture content θ (%) and matric tension ψ_m (cm) for each time and depth.

Figure B-22 presents these K values for each horizon plotted against the matric tension. A logarithmic scale is used on the vertical axis to contain the wide range of values achieved over the small range of tensions.

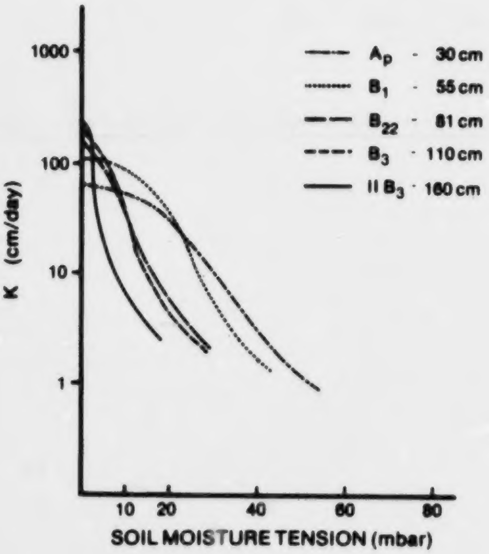


Figure B-22. Hydraulic conductivity values determined with the instantaneous-profile method (Bouma et al., 1974a).

TABLE B-2. CALCULATION OF SOIL MOISTURE FLUX

Time (days)	Z (cm)	$\frac{\partial \theta}{\partial t}$ (days^{-1})	$(\frac{\partial \theta}{\partial t})dZ$ (cm/day)	$q = \Sigma(\frac{\partial \theta}{\partial t})dZ$ (cm/day)
0.0	30	1.70	51.0	51.0
	55	1.75	43.8	94.8
	81	1.70	44.2	139.0
	110	1.15	33.4	132.4
	160	0.42	21.0	193.4
0.15	30	1.50	45.0	45.0
	55	1.70	42.5	87.5
	81	1.65	42.9	130.4
	110	1.10	31.9	162.3
	160	0.40	20.0	182.3
3.0	30	0.10	3.00	3.00
	55	0.30	7.50	10.50
	81	0.18	4.68	15.18
	110	0.18	5.22	20.40
	160	0.12	6.00	26.40
6.8	30	0.04	1.20	1.20
	55	0.08	2.00	3.20
	81	0.05	1.30	4.50
	110	0.04	1.16	5.66
	160	0.14	7.00	12.66
12.0	30	0.04	1.20	1.20
	55	0.02	0.50	1.70
	81	0.04	1.04	2.74
	110	0.04	1.16	3.90
	160	0.08	4.00	7.90
18.0	30	0.04	1.20	1.20
	55	0.02	0.50	1.70
	81	0.03	0.78	2.48
	110	0.04	1.16	3.64
	160	0.06	3.00	6.64

TABLE B-3. CALCULATION OF HYDRAULIC CONDUCTIVITY

Z (cm)	q (cm/day)	$\frac{\partial H}{\partial Z}$ (cm/cm)	K (cm/day)	θ (%)	ψ_m (cm water)
30	51.0	1.00	51.0	31.7	--
	45.0	0.86	52.3	31.5	- 9
	3.00	0.65	4.62	30.1	-36
	1.20	0.63	1.90	29.7	-44
	1.20	0.53	2.26	29.4	-50
	1.20	0.63	1.90	29.1	-57
55	94.8	1.00	94.8	39.4	+26
	87.5	0.86	101.7	39.0	- 7
	10.5	0.65	16.15	28.6	-25
	3.20	0.63	5.08	27.7	-31
	1.70	0.53	3.21	27.3	-33
	1.70	0.63	2.69	26.9	-38
81	139.0	1.00	139.0	34.0	+29
	130.4	0.86	151.6	33.7	0
	15.2	0.65	23.4	29.5	-11
	4.50	0.63	7.14	28.9	-19
	2.74	0.53	5.17	28.5	-18
	2.48	0.63	3.94	28.4	-21
110	172.4	1.00	172.4	33.7	+25
	162.3	0.86	188.7	33.4	+15
	20.4	0.65	31.4	30.3	+ 5
	5.66	0.63	8.98	29.8	-15
	3.90	0.53	7.36	29.7	-16
	3.64	0.63	5.78	29.7	-18
160	193.4	1.00	193.4	30.0	+11
	182.3	0.86	212.0	--	+ 2
	26.4	0.65	40.6	31.4	+ 9
	12.66	0.63	20.10	--	- 3
	7.90	0.67	11.79	30.5	- 7
	6.64	0.81	8.19	30.2	- 6

Double tube method for in situ saturated hydraulic conductivity--The double tube method can measure saturated hydraulic conductivity for horizons within a soil profile. It is similar to a well test under special conditions. The method is not quick or simple, and it requires specialized equipment. The saturated hydraulic conductivity measured is not in the vertical direction. It is the resultant of all directional components, thereby reducing the application of the data to many real world situations. It is a useful method for ranking K_{sat} among soils.

Principles and procedures--For this method, an auger hole is dug to the soil horizon where permeability is to be measured. The diameter of the hole must be larger than that of the outer tube as shown in Figure B-23. The bottom of the hole is made level, cleared of loose debris, and covered with 2 cm of coarse sand. The two concentric tubes are installed so that their lower edges cut into the soil to a depth of 2 to 4 cm. The cover plate with standpipes is attached to complete the apparatus. The diameter of the outer tube must be at least twice that of the inner tube (Bouwer, 1962).

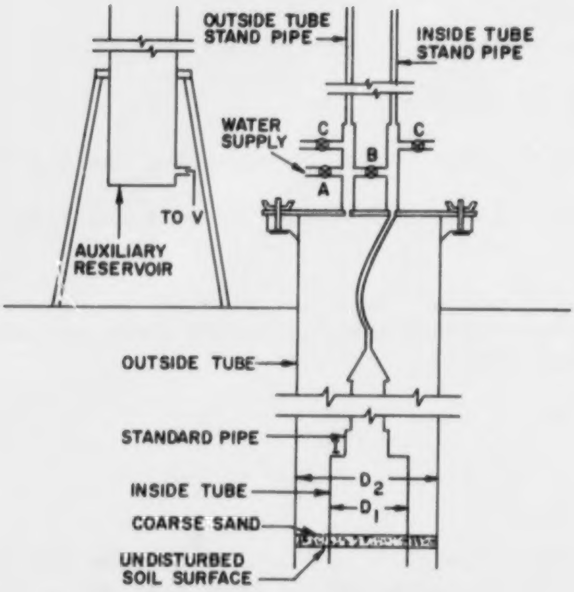

OUTSIDE TUBE STAND PIPE
INSIDE TUBE STAND PIPE
WATER SUPPLY
C
C
B
A
TO V
AUXILIARY RESERVOIR
OUTSIDE TUBE
STANDARD PIPE
INSIDE TUBE
D2
D1
COARSE SAND
UNDISTURBED SOIL SURFACE

Figure B-23. Schematic diagram of equipment for the double-tube method in place (Boersma, 1965).

Water is applied, maintaining equal head in both tubes. When valve B is open, the two concentric systems are interconnected; when the valve is closed, they are isolated from each other. Water is applied for considerable time to saturate the surrounding soil. When this has been achieved the test can begin.

Two measurements are made, and from these results, the hydraulic conductivity is calculated. First, the water is forced into both systems until the two standpipes overflow. The water supply is cut off and valve B is closed. The water levels in the standpipes will begin to fall. By manipulating valve C in Figure B-23, the outside level can be made to fall at the same rate as the level in the inner tube. The time required for the water level to drop past calibration marks on the inner tube are recorded on a series of stopwatches. In the second experiment the water level is maintained at the top of the outer tube standpipe while the level in the inner pipe drops at its own rate. Again time intervals are measured. A second person with stopwatches is required for these recordings.

The hydraulic conductivity can be calculated from (Bouwer and Rice, 1964)

$$K_{sat} = \frac{R_s^2}{FR_I} \cdot \frac{\Delta H}{\int H dt}$$

where R_s = radius of the inner tube standpipe

R_I = radius of the inner tube

F = a dimensionless factor encompassing the geometry of the apparatus

The F for a particular apparatus can be found from standardized curves (Bouwer, 1962).

The results of the two tests are plotted, as shown in Figure B-24. These curves generally approximate straight lines where applying the following expression (Bouwer, 1964b)

$$\frac{\Delta H}{\int H dt} = \frac{2\Delta t}{t^2_{eq.lev.}}$$

Here ΔH is the difference in its head for the two measurements at a given time and $\int$Hdt is the magnitude of the head loss until that time, t equal levels. The right hand quantity can be calculated easily from the time interval data and the result substituted into previous equations for K_{sat}

Comments--The double-tube method yields a value for hydraulic conductivity only under saturated conditions. However, this conductivity is poorly defined being some resultant of the vertical and horizontal conductivities of the soil at the sample point. Because of this fact the numbers obtained by this method cannot be used directly for calculation of infiltration rates or drainage field size. This relegates the technique to the role of simply ranking the saturated permeabilities of soils.

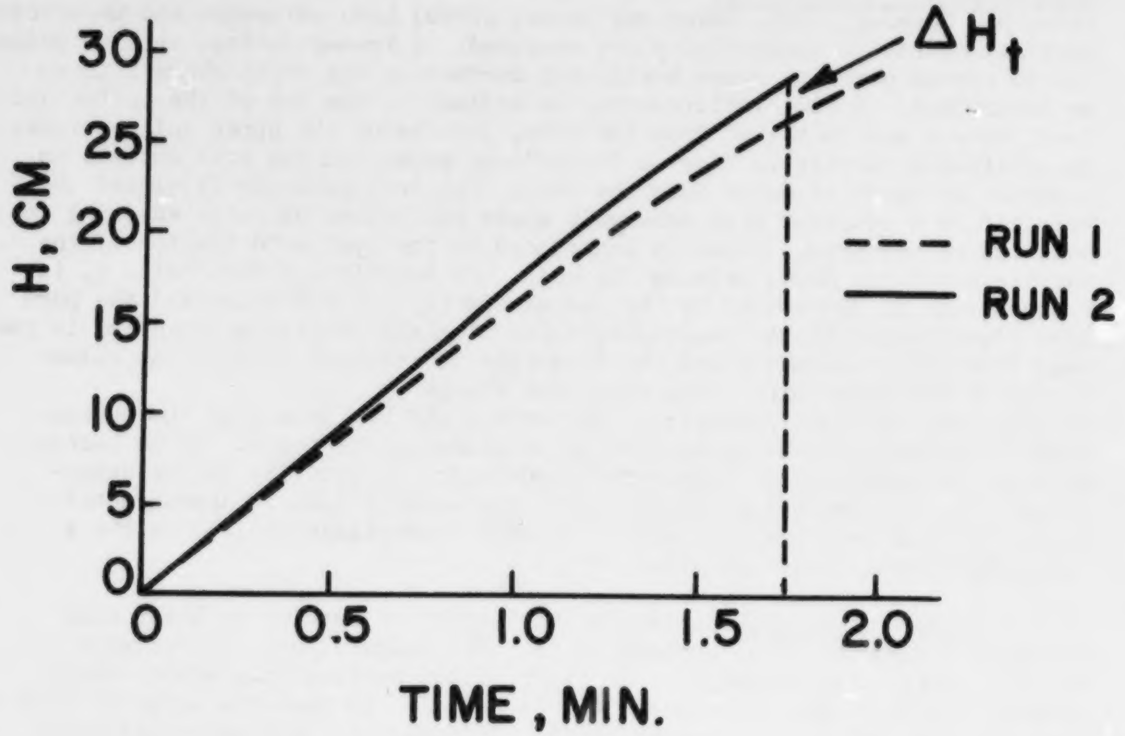


Figure B-24. Graph showing the results of the observations necessary to calculate the hydraulic conductivity obtained by the double-tube method (Boersma, 1965).

The double-tube test was developed for use on sandy soils, where isotropic conditions were assumed. The dimensions of the apparatus (Bouwer, 1962) were experimentally adjusted for these soils and may be inadequate for use on soils with well-developed soil structure, where horizontal and vertical components of hydraulic conductivity may differ greatly.

K values obtained with the double-tube method are conservative estimates when compared to the results of other methods. For example, the mean K_{sat} of the Plano silt loam, B22t horizon is 138 cm/day when measured with the double-tube method (Bouma, 1971). The crust test method yields a mean value of 610 cm/day.

In summary, the test is complex and requires special equipment. The data obtained can be used to rank soil permeability but not for direct estimation of drainage field size.

The crust test method for in situ measurement of saturated and unsaturated hydraulic conductivity--In the crust test method (Hillel et al., 1970; Bouma and Denning, 1972; Baker and Bouma, 1976b) both saturated and unsaturated vertical hydraulic conductivity are measured. A free-standing, in situ pedestal is carved with its upper horizontal surface at the depth where K is to be determined. A ring infiltrometer is affixed to the top of the column and tensiometers are installed from the side, just below the upper soil surface. An artificial barrier to flow--a "crust"--is placed on the soil surface to restrict movement of water into the soil. The infiltrometer is closed and attached to a constant head reservoir where the volume of water entering the soil can be measured. Water is introduced to the system on the top of the barrier and moves down, wetting the soil. An equilibrium flow rate, q, is reached that is determined by the resistance (r_b) of the crust and the pore size distribution of the underlying soil. The soil moisture potential is then read from the tensiometer and the flow rate is measured directly as volume change at the reservoir. This flow rate change at that soil moisture potential. By varying the resistance of the barrier, other conductivities at corresponding tensions can be found. If no barrier is used, unrestricted or "saturated" hydraulic conductivity can be determined. The corresponding tension under these conditions is approximately zero. In this way, several points (tension-conductivity pairs) on the K curve for that soil are obtained.

Principles involved--An artificial "crust" or barrier to water flow located at the horizontal surface of the soil pedestal acts to restrict entry of water into the soil. The fewer, finer pores of the crust cannot conduct liquid to the underlying soil as rapidly as when the water is able to flow downward through the natural soil. This results in a water-saturated crust with water standing on its upper surface. The soil is not saturated if the saturated conductivity of the soil is greater than that of the barrier. This is true as long as a water table or another restrictive layer do not interfere by their presence, causing the hydraulic gradient to differ greatly from unity.

Under steady state flow,

$$q_b = q_{soil}$$

and

$$K_b\left(\frac{dH}{dZ}\right) = K_{soil}\left(\frac{dH}{dZ}\right)$$

where q_b and q_{soil} are the flow rates or fluxes through the barrier and soil, respectively, and (dH/dZ) is the hydraulic gradient in each case.

The gradient in the soil during steady state flow approximates unity, with gravity as the major force. For the crust test then,

$$q_{soil} = K_{soil}(\psi_m) = K_b(\frac{dH}{dZ})$$

By measuring q_{soil} and the corresponding soil moisture potential (ψ_m), a potential-conductivity pair is defined for each barrier used.

Installation of equipment--The crust test measures the vertical hydraulic conductivity of a volume of soil from a horizontal plane downward. For this purpose, a free-standing pedestal of soil is carved out in situ, its undisturbed base continuous with underlying soil horizons. Water entering the upper surface of the soil is restricted to essentially downward flow by the sidewalls of the pedestal. Because the pedestal is still attached to underlying horizons, pore-continuity is assumed to be maintained and water movement representative of natural soil conditions.

A small pit is dug at the site where K measurements are to be made and a ledge is formed on one side at the horizon of interest. Care must be exercised to avoid compacting this horizontal surface for this is where infiltration occurs during measurements. This level can be approached by the use of a shovel. Final preparation of the infiltrative surface is done with a knife or spatula by either scraping or lifting out portions of the soil. It is best to cut downward about 1 cm with the blade of the knife and then lift and move it to one side at the same time. This will loosen a fragment of the surface. By taking only small bits (2 to 3 cm wide) at a time and by holding the loosened soil against the blade as it is raised, a suitable surface is created. Practice is required before this becomes efficient, but it does leave a fresh horizontal soil surface with no smearing. In sandy soils, this method of surface preparation may not be necessary.

Infiltrometers are 24-cm inside diameter (see Figure B-25). Thus, the area enclosed is about 490 cm^2, and in structured soils, includes many soil peds, channels, and pores. It is sufficiently large to accomodate several networks of pores and not truncate meandering flow pathways that involve some lateral movement. This is especially important for conductivities at or approaching saturated conditions. The minimum diameter that will yield relatively accurate data is about five times the width of the peds in the soil. In sandy soils, small areas could be used but this practice may not include the influence of biopores at or near saturation. For these reasons, a minimum infiltrometer diameter of 15 to 20 cm is advised. Diameters greater than 40 cm are difficult to work with but may be necessary for special conditions. A comparison of 24 cm and 48 cm diameter rings indicates no measurable difference in mean unsaturated hydraulic conductivities for medium, subangular blocky silt loams and for sandy soils, but the variability of results on the larger samples is somewhat lower (Baker, 1976d). However, for K measurement at or near saturation ($\psi_m \rightarrow 0$), a significant difference was measured for pedestal diameters of less than 24 cm.

Next, a free-standing soil pedestal is carved down several centimeters to facilitate placement of the ring infiltrometer on the pedestal. At first, the pedestal is made four or five centimeters larger in diameter than

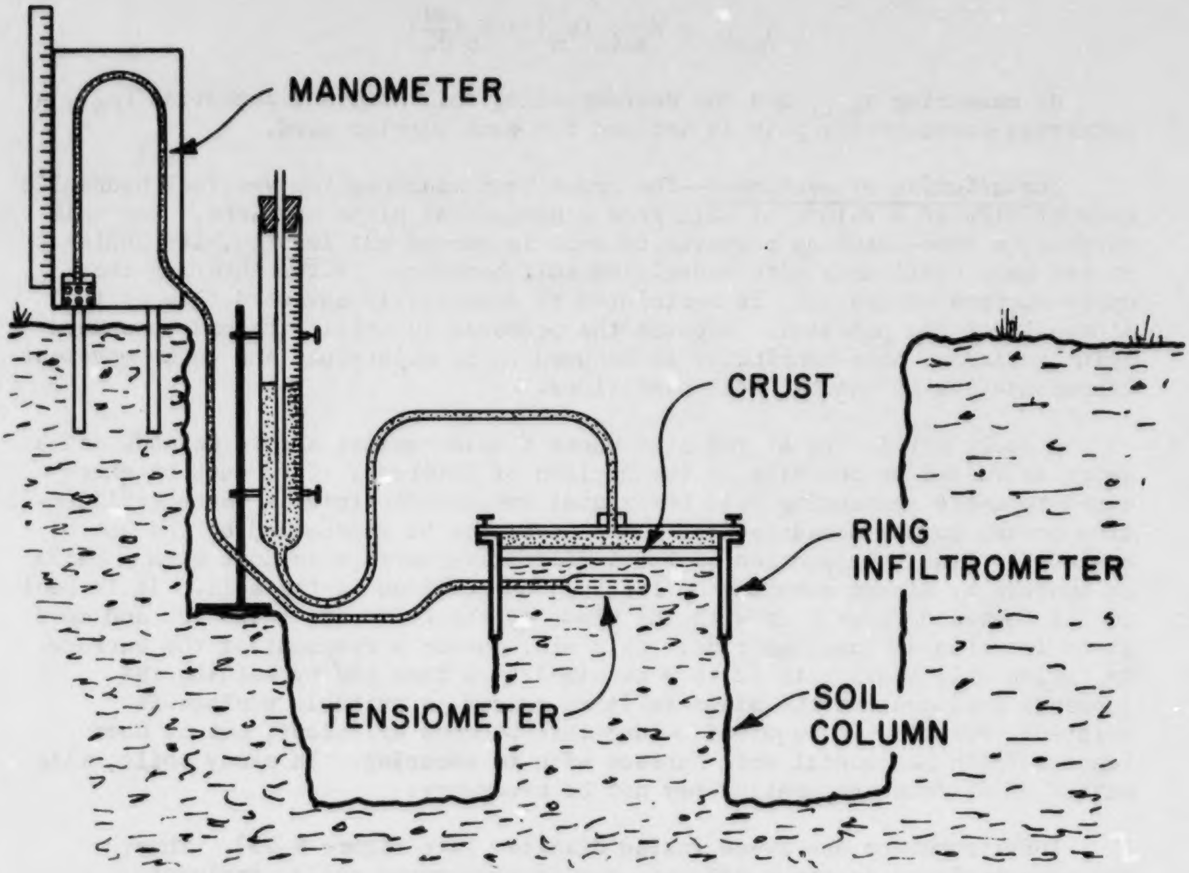


Figure B-25. Schematic diagram of the crust-test procedure.

the ring. A strong-bladed hunting knife works well for this purpose. The infiltrometer is placed on top of the pedestal and then carefully pushed into the soil. By grasping the ring with both hands and gradually applying weight, the sharp edge of the ring will part the soil allowing a good fit. Excess soil around the outside of the ring can be cut away with the knife. The ring is pushed down until only about 1.5 centimeters of sidewall remains above the soil. This space is required later in the procedure for the barrier and water. A small gap may appear between the soil and the side of the ring but is no cause for concern. Under unsaturated conditions, water will not flow along this void and as the soil becomes wetter it will expand to seal the gap. Therefore, at saturation, no peripheral void will exist.

The soil pedestal must be carved to a height of at least one ring diameter. For a 24 cm diameter ring, a height of 30 cm is recommended. Experiments have shown that differences in saturated hydraulic conductivity due to the height of the soil pedestal can be large for low pedestal heights (Baker, 1976d). K_{sat} values measured for a pedestal 15 cm high were 20 percent higher than those measured at 30 cm height. If, however, heights of 30 or greater are used, no significant difference in K values is detected. This was found to be true for both structured (silt loam) and nonstructured (sandy) soils.

Carving is best carried out again with the knife or a hand trowel. Use of a shovel in close quarters may easily result in the cracking or breaking of the soil pedestal. The pedestal should be of uniform diameter (± 1 cm) over its entire length. When complete, it is wrapped in aluminum foil to reduce evaporation from sidewalls. This covering remains in place for all unsaturated measurements and is replaced by a quick-setting plaster coating for saturated K measurements.

Tensiometry--Small diameter (4.5 mm) porcelain tensiometers are used to measure soil moisture potential within the soil pedestal. These instruments have a tee configuration (see Figure B-26). The porcelain cup can have pore sizes of about 0.25 to 1.00 . Pore sizes greatly outside this range will affect the response time and air entry value of the tensiometer. The porcelain cup is attached to 3-mm (1/8 inch) flexible plastic tubing which runs to one arm of a brass tee fitting. The tubing on the other arm of the tee is fitted with a brass screw cap assembly. Fine nylon tubing then runs inside from the cap, through the tee to the very tip of the cup. This is used for filling the tensiometer to purge the cup of entrapped air. A third plastic tube extends from the base of the tee to the manometer board where the free ane d is submerged in the mercury reservoir.

Before placement of the tensiometer cup in the soil column, a hole is augered horizontally into the soil pedestal from the side to a point just beneath the center of the soil surface. The diameter of the augered hole is slightly less than that of the porous cup so that a snug fit assures good contact with the soil. After the cup is in place, the hole should be sealed with mud or glue to prevent evaporation. Tensiometers should be placed at a depth of 2 to 5 centimeters under the infiltrative surface. Holes can be made in the sidewall of the infiltrometer to facilitate this. Earlier in the development of the crust test, two tensiometers were placed in series, one 2 cm below the other so that the hydraulic gradient could be measured (Hillel et al., 1970). This practice is not always practical. Two independent tensiometers can be used to yield the average potential of the soil. This is useful because a tensiometer generally registers the tension of only the soil in its immediate vacinity. Sometimes this can be misleading.

In setting up the apparatus tensiometers are arranged as shown in Figure B-27 where the free end of the tensiometer runs through a manometer board to a mercury reservoir. A graduated scale is used to read potential as centimeters of water when mercury rises on this scale. The difference in elevation between the porous cup and the surface of the mercury in the reservoir is referred to as the correction factor or CF. This is easily

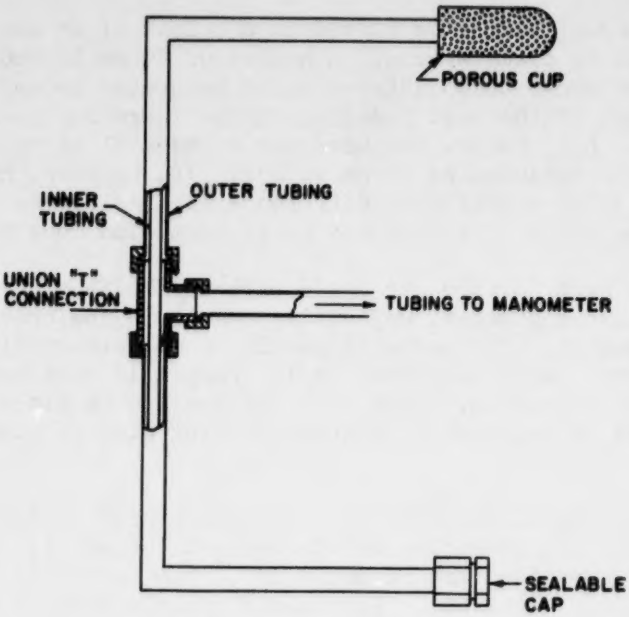

POROUS CUP
INNER TUBING
OUTER TUBING
UNION "T" CONNECTION
TUBING TO MANOMETER
SEALABLE CAP

Figure B-26. Schematic diagram of tensiometer, showing major components of the system.

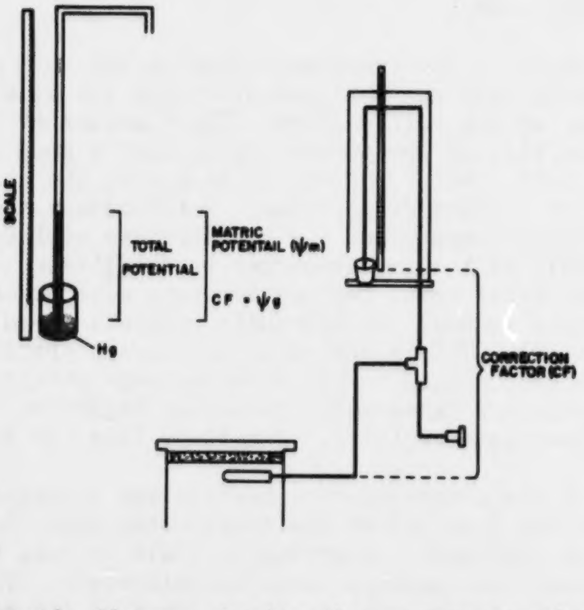

SCALE
TOTAL POTENTIAL
MATRIC POTENTAIL (ψm)
CF • ψg
Hg
CORRECTION FACTOR (CF)

Figure B-27. Schematic diagram showing the components of potential (ψ_m and ψ_g) and the measurement of correction factor (CF) on the experimental apparatus.

measured in the field with two meter sticks and a level. It is subtracted from the total tension to yield matric soil tension.

Once installed properly, the tensiometer is filled with deaired water using a 50-ml syringe with a 22-gauge needle. The needle is inserted in the end of the fine inner tubing at the capped end and water is forced through to the top of the porcelain cup. A wetting front will proceed along the space between the inner and outer tubes from the cup through the tee and out the capped end. If the outer tube is held tightly against the butt of the needle, water can be forced up the third branch to the mercury reservoir. Continued pressure will drive water out at the mercury cup, purging the system of all air. The needle is withdrawn and the end is capped. Before inserting the tensiometer cup firmly into the soil, the cup is wiped to remove excess water.

The flux measurement--Measurement of the rate of water flow through the resistant barrier is accomplished by a burette system. A flexible plastic tube connects the burette to a large port at the center of the plexiglass cover plate where it is attached by a threaded brass fitting (see Figure B-25). When the system is closed, i.e., when the cover plate is sealed onto the ring infiltrometer, water can be introduced to the system via the burette. A large hydraulic head is desired to maintain steady-state flow. This is easily achieved by use of a Mariotte tube, consisting of a rubber stopper that fits the burette with a small tube running through it. This inner tube extends to the base of the burette and the vertical height that it is raised above the crust surface is the effective hydraulic head in centimeters (Figure B-25). Flux is measured as volume change in the burette. A correction must be made for the amount of water displaced by the mariotte tube. Several burettes of different sizes should be available as they may be needed for certain conditions. The accuracy of any flow rate can be improved by using a more closely graduated burette.

The resistant barrier or "crust"--A relatively quick-setting, dense gypsum is used in the barrier or "crust." It is mixed with just enough water to make a thick paste, which is then spread upon the infiltrative surface to a thickness of 0.5 to 1.5 cm. This is easily done with the bare hand. Care must be taken to tamp the paste near the edges to achieve good contact with the metal ring. This contact is the site of most leaks through the barrier.

The resistance of the barrier to water flow can be regulated by mixing various proportions of sand and water into the gypsum before making a paste. The presence of the sand in the crust causes larger pores to be formed thus allowing more water to flow through. Higher sand content in the mixture causes less resistance and allows higher rates of flow through the crust. Proportions of sand and gypsum are measured by volume percent in a 1-liter graduated cylinder. The proportion is expressed as the percentage of gypsum in the crust. A 25 percent crust is one quarter gypsum by dry volume. The dry sand and gypsum should be mixed thoroughly in a dishpan before water is added.

Generally, the first crust to be used is the most resistant one. Succeeding crusts are progressively less resistant to flow. This allows the equilibrium flow rate to be reached by wetting, yielding the wetting K curve. A similar but slightly higher curve will be obtained if equilibrium is reached by drying, due to hysteresis. For this reason, it is important to know the soil moisture potential before introducing water to the system. If a crust leaks or there is some other malfunction, the soil must be allowed to drain before reinitiating the experiment.

Removal of the crust after its steady state flow has been reached is sometimes a difficult task. A 100 percent gypsum crust is very tough, requiring chiseling action by a sharp implement such as a screw driver. The first one or two pieces will shatter in the process, but following ones are more easily removed. Excessive pounding may damage the soil pedestal. Crusts of less than 75 percent gypsum are much easier to remove and can be handled with a knife as used when preparing the fresh soil surface. This is the crudest technique in the procedure and requires some practice.

Other types of barriers have been used with some success; noteably work with synthetic foam sponges that rest on the soil (Baumer et al., 1976). Few types of easily changed barriers achieve as good contact with the soil as does a gypsum crust.

Measurement procedure--Once a site is instrumented, the crust is selected so that its resistance to flow will yield the approximate soil tension that is desired. This is based on a general knowledge of the soil's K curve (Figure B-28). The tension induced by the crust must be somewhat lower than the starting soil tension in order to achieve wetting of the soil. If the soil is quite dry to begin with, a 100 percent crust is usually used first to yield tensions in the 60 to 100 cm range. For higher tensions, silt loam or clay can be smeared on the surface of a 100 percent crust.

After the crust has hardened, the gasket and cover plate of the ring infiltrometer are bolted into place, so that the air escape port is at the highest position (Figure B-25). The burette assembly is attached to the central port and water is introduced. Care should be taken with low gypsum content (light) crusts that a piece of aluminum foil or other material is placed directly beneath the central port for water entry. This will prevent incoming water from eroding a hole in the crust by impact. Water flows in under a hydraulic head to speed filling of the space between the crust surface and the cover plate. The air escape port is open until all air has been purged from the system. At this point, the Mariotte tube and stopper are placed into the burette to reduce the hydraulic head, and the air escape port is capped. The air escape port is opened whenever the Mariotte device is unstoppered, otherwise sudden changes in head and back-pressure from below the crust can cause leaks to occur. When air bubbles begin to rise from the tip of the Mariotte tube, the first volume measurement can begin.

Volume measurements are recorded periodically until equilibrium is reached. Simultaneous tension measurements are also made. These values will change with time as shown in Figure B-29. Equilibrium is reached as the values approach

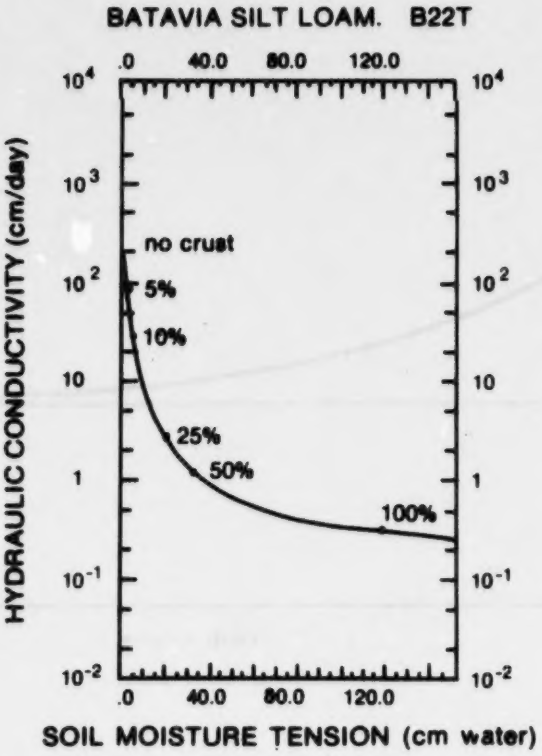


Figure B-28. Hydraulic conductivity curve measured at a site in the B22t horizon of the Batavia soil series.

an asymptote. The conductivity at that tension is calculated from the measured constant flow rate.

After equilibrium is reached for the first crust, it can be removed. This is done by opening the air port, removing the plate and gasket and soaking up any standing water with a sponge. The old crust is then taken off and replaced by a less resistant one. The procedure is repeated. Each succeeding crust yields a point on the K curve. Figure B-28 exemplifies this for a Batavia silt loam soil, showing the results of a series of crusts and the K curve derived from these points. Each point represents the potential-conductivity pair caused by a particular crust. The percentage indicated by each point is the gypsum content of the crust that was used to obtain that point.

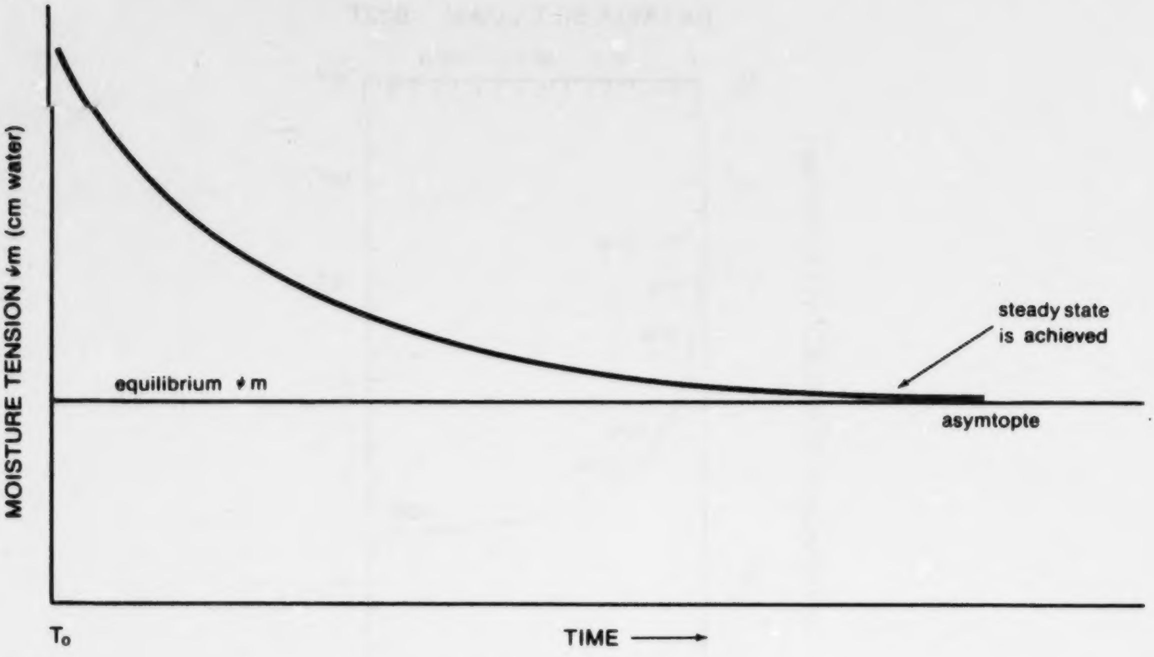


Figure B-29. Generalized graph of the rate of decrease of matric tension (ψ_m) as equilibrium is approached by wetting of the soil.

Saturated hydraulic conductivity is measured using the same apparatus with one modification: the aluminum foil cover is replaced with a coating of quick-setting dental-grade plaster. The dental plaster is mixed in a dishpan to be very fluid and is poured or splattered against the sides of the soil pedestal until the plaster is more than 3 mm thick. This seals the soil to prevent water from flowing directly out the sides. Care is also taken that any extra holes in the sides of the metal ring are sealed with plaster or glue. Leakage of this sort is not a problem during unsaturated flow conditions when soil moisture tension retains water in the soil pores.

For saturated flow measurements, no crust is used. If there is no barrier to flow, all or nearly all pores will conduct liquid giving a reasonably accurate measure of the maximum flow through the soil. The procedure is conducted as for unsaturated conductivity measurements. Tensions are at or very near zero. Volume change is recorded until it is relatively stable to yield the equilibrium flow rate. Water flow should not be maintained for more than 45 minutes or an hour. Appreciable changes in the soil pores will occur after this time.

Trouble shooting--If problems occur that have no obvious cause, they can be traced by examining the three basic subsystems of the test equipment. These are the crust, the tensiometers and the water flow systems.

Probably the first thing to look for in a crust is a leak. This is a relatively simple procedure. Push down firmly on the center of the plexiglass cover plate of the infiltrometer and then release quickly. This decreases and then increases the volume over the crust. To compensate for the volume change, water must flow in via the burette, but this takes time. If there is a leak in the crust, it is easier for air to bubble up from beneath the crust. Therefore, if bubbles suddenly appear, a leaky crust has been demonstrated, and it can be replaced. Data collected from a leaky crust should be discarded. It is perhaps a good idea to conduct this test on all crusts soon after water is applied.

Problems may occur in the flow measurement system. These may appear as a lack of flow or as unbelievable flow rates. The first thing to look for is a leak in the tubing or at the Mariotte stopper. Air bubbles of any size in the system can also hinder flow, and they are prone to temperature effects, causing inconsistent and inaccurate results. Also, fluctuations in the temperature of the water in the system can lead to other inaccuracies. On sunny days or partly cloudy summer days, the direct heating of the sun's rays can be quite dramatic, and the burette, tubing and cover plate must be covered with aluminum foil to reflect most of the unwanted heat.

It is best to use two tensiometers so that if one should fail, the experiment is not interrupted. All tensiometers should be tested and thoroughly examined before they are taken to the field. This will save much time and frustration on site. A good tensiometer can be used many times once it has proven itself. The usual cause of problems is a leak allowing air to enter into the system. Refilling the tensiometer with deaired water may be a temporary solution. If the cause is not apparent, it is best to replace the tensiometer entirely with a spare. Periodically, tensiometers should be retested in the laboratory.

Data analysis--When a final volume measurement of steady-state flow is obtained, it can be used for calculation of the soil's hydraulic conductivity at that tension. The volume of flow per minute is converted to centimeters per day, by the following formula:

$$K\left(\frac{cm}{day}\right) = flow\left(\frac{ml}{min}\right) \cdot 1440\ \frac{min}{day} \cdot \frac{1}{Area\ (cm^2)}$$

where the area is that of the infiltrative soil surface. For a 25 cm diameter ring this area is 490 cm^2. Thus a flow rate of 30 ml in 30 minutes yields:

$$K = \frac{30 \text{ ml}}{30 \text{ min}} \cdot 1440 \frac{\text{min}}{\text{day}} \cdot \frac{1}{490 \text{ cm}^2} = 2.94 \text{ cm/day}$$

at the equilibrium soil tension measured.

It is best to do the calculations before changing the crust. It is possible to detect erroneous data and then by double-checking the subsystems of the equipment, to eliminate the cause.

Advantages and disadvantages of the technique--The main advantage to the use of the crust test is the direct and geometrically simple pattern of vertical water movement that is measured. Also, this is accomplished in situ without major disruption of natural soil pores. Most methods for hydraulic conductivity measurement, especially at saturation, involve some component of lateral if not radial flow. These involve more complex mathematics and describe flow patterns that are not common to most real situations. The pedestal in the crust test is also isolated so that external interference is held to a minimum, unlike the instantaneous profile method. Another major advantage of the use of crusts is that a range of unsaturated conductivities can be obtained without as large a time commitment as some other methods. Also, the amount of variation in the technique itself is much lower than that of other methods.

Disadvantages of the method are that it is complicated, although not nearly as complex as the double tube test is for a single conductivity. It requires that work be carried out in the field and this means the work is subject to weather limitations. The technique only measures vertical hydraulic conductivities.

Comparison and discussion of methods--In choosing which permeability measurement method to use, several factors must be considered. First, the dimensions of soil permeability to be measured must be defined, and a procedure must be selected that truly measures that permeability. Second, it must be decided how accurate the measurements of permeability must be. The third consideration is whether the procedure is efficient for the given purpose. In most cases some compromise is made between these three aspects of the problem. Each of these factors will be discussed with respect to the five above-mentioned permeability measurement methods and their application to liquid waste disposal.

Soil hydraulic conductivity (K) in the vertical direction (K_v) is required for most wastewater applications. The importance of vertical movement is due to the dominance of gravity flow in most waste disposal systems. Horizontal conductivity (K_h) may be more important for narrow subsurface trench designs and in situations where downslope movement of effluent is anticipated. If the expected steady state flow rate through a stable clogged soil layer is known, then the required drainage field surface area can be calculated directly. The steady-state flow rate is K_{vss} at ψ_{mss}, the soil moisture potential at steady-state conditions. This means that K must be known for particular unsaturated conditions that will be dictated by the clogging layer. A useful method would be one that estimates K_{vss}.

Few of the procedures as described are capable of this measurement. The percolation test measures a radial combination of vertical and horizontal conductivities at saturation only. Neither K_v at saturation nor unsaturated K can be determined. The test serves only as a rough ranking measure, simply indexing soil permeability. Soil core methods can measure a vertical conductivity, but these values are a function of core length (Anderson and Bouma, 1973) and exhibit high variation at small dimensions. The measured K_v may not be applicable to field situations. The double-tube test measures only saturated conductivity. K_{vsat} can be determined only by the use of a more complicated procedure (Bouwer, 1964a) than the one described here. Both the instantaneous profile method and the crust test measure K_v for unsaturated conditions. For determinations at a specified horizon, the crust test may be simpler to use.

The accuracy and reproducibility of a few of these methods has not been established. The coefficient of variation (Cv) of measured conductivities is a convenient means of comparison of within test variation. For the percolation test Bouma (1971) reported Cv = 54 percent for the falling head method which is commonly used, and Cv = 35 percent for the constant head procedure. Hill (1966) in Connecticut found similar values of Cv = 49 percent and Luce (1973) reported Cv = 38 percent for soils in Iowa. Anderson and Bouma (1973) found Cv = 66 percent for K measurements made on 10 cm long soil cores. While comparing two field procedures for measurement of K_{sat}, Bouma (1971) found that the double-tube test has Cv equal to 25 percent. Nielsen et al. (1973) reported mean Cv of 86 percent at saturation, and average coefficient of variation of 381 percent for soil at 60 percent saturation.

The method that is simplest and easiest to use, while still providing useful information, usually finds the most successful field application. Of the two techniques that can measure K_v for unsaturated soil, the crust test is probably the least time-consuming. It requires little specialized or expensive equipment, and only a few liters of water. The instantaneous profile method requires expensive equipment and a much longer time period to collect the data. A skilled technician is capable of carrying out either procedure.

Selection of a method of conductivity measurement should be based primarily on the suitability of the measured values for the particular application. Then the feasibility of the procedure must be balanced against these requirements. The percolation test is a good example of a test that is used because of convenience, long after it was realized that it is not an acceptable measure of soil permeability.

Variability of Hydraulic Conductivity in Soils--

Soil variability has been a topic of interest among soil scientists for some time. This interest has been directed largely toward morphological

(McCormack and Wilding, 1969) and chemical variations in the soil that led to problems in classifying soils and also to nutrient variations (Beckett and Webster, 1972) that influenced agronomic uses of the land. The variability of physical properties has also been the subject of numerous investigations (Mason et al., 1957) especially with respect to engineering and flood control applications (Rogoswki, 1972). The variability of hydraulic conductivity is of great importance for on-site liquid waste disposal (Bouma, 1973). Knowledge of the expected range of conductivities or of the expected minimum conductivity for a given site would facilitate the use of soil maps and soil series identification, rather than the now almost exclusive reliance on the testing of each and every site.

By defining the conductivity curve and its corresponding variation for each of several soil series, it may be possible to compare and contrast these series on the basis of their hydraulic conductivity characteristic. Series with similar K characteristics, for all practical purposes, could be grouped together into arbitrary hydraulic conductivity classes to facilitate use of this information. This would result in a special use classification, designed for application to waste disposal planning and design.

Recent work has focused on the variability of soil hydraulic conductivity in the field (Mason et al., 1957; Nielsen et al., 1973; Baker and Bouma, 1976b). Current work by Baker (1976a) has dealt with variability of K in nine soil series, the morphology of which range from single-grained structured sands to heavy clay loam soils having well-developed structure. The results of this study will be summarized briefly here.

Soils studied and site selection--The variability of K of nine soil series in Wisconsin were studied. These soils were selected to span a wide range of textural, structural and hydraulic properties. They were the Batavia Boone, Hochheim, Magnor, Morley, Ontonogon, Plano, Plainfield and Withee series, as described by the National Cooperative Soil Survey. Textural and structural information for each series is summarized in Table B-4. The soil horizons in which conductivity was measured are also listed there. Generally, the horizon with lowest permeability was selected for measurement since these layers represent the limiting conditions for wastewater flow.

The genesis of these soils is of interest since the nature and subsequent environmental adaptations of the initial material strongly influence some morphological properties of the soil (Boul et al., 1973). The sum of these morphological properties is used to classify soils into groups and series and some of the physical characteristics of soils are expressions of these morphological properties. Detailed morphology of these soils can be found in soil profile descriptions published by the National Cooperative Soil Survey. The Plainfield series developed largely in glacial outwash in central Wisconsin. Although it does not offer limitations to on-site waste disposal due to permeability, it has been included in this study as a landmark soil. The Boone series was formed in residual sand deposits eroded from Cambrian sandstones in the hilly driftless area of Wisconsin. The Magnor and Hochheim soils formed in loamy glacial till covered by a thin loess cap. Both soils present waste disposal problems, especially the Hochheim which has weak structural integrity. The Batavia and Plano series developed in deep (1 to 2 meter

TABLE B-4. TEXTURAL AND STRUCTURAL INFORMATION FOR SOIL SERIES USED IN HYDRAULIC CONDUCTIVITY VARIABILITY STUDIES

Row no.	Soil identification	Horizon selected	Mean depth to sample point(cm)	Texture	Structure	Frequency of biopores
1	Withee Series fine-loamy,mixed,frigid Aeric Glossaqualf	IIB2	75	loam	moderate,coarse to medium subangular blocky	few medium common fine
2	Ontonogon Series very fine,illitic,frigid Typic Eutroboralf	B2	60	heavy loam	moderate,medium angular blocky	few coarse, common medium
3	Magnor Series fine-loamy,mixed Aquic Glossoboralf	IIB	75	heavy loam	moderate,medium subangular blocky	few coarse, common medium
4	Hochheim Series fine-loamy,mixed,mesic Typic Argiudoll	C	50	loam	weak,coarse, blocky,becoming massive	few coarse, few medium
5	Plano Series fine-silty,mixed,mesic Typic Argiudoll	B2	90	silty clay loam	moderate,medium subangular blocky	few coarse, common medium and fine
6	Batavia Series fine-silty,mixed,mesic Mollic Hapludalf	B2	75	silty clay loam	moderate,medium subangular blocky	few coarse, common medium and fine
7	Morley Series fine,illitic,mesic Typic Hapludalf	IIB3	90	heavy,silty clay loam	moderate,medium prismatic, to coarse,blocky structure	few coarse, common medium
8	Boone Series sandy,mesic,uncoated Typic Quartzipsamment	C	60	fine sand	structureless	few medium
9	Plainfield Series sandy,mixed,mesic Typic Udipsamment	C	75	medium sand	structureless	few medium

thick) loess deposits overlying glacial till. The Batavia formed under forest vegetation generally and the Plano under prairie. The Morley series formed in loess and fine, calcareous glacial till in southeast Wisconsin. Long wet periods during the spring and early summer limit wastewater disposal in this soil. The Withee series chiefly developed from lacustrine sediments left by glacial Lake Wisconsin. Finally, the Ontonogon was formed from lacustrine deposits on the shore of Lake Superior. This last soil series is grouped as one of the notorious "red clays" that have prevented development of large areas of land in some northern counties of Wisconsin because of their low permeability.

Detailed soil maps in published soil survey reports (National Cooperative Soil Survey), showing mapping units as small as 0.8 ha (2 acres) in area, were used to select experimental sites in 13 counties of Wisconsin. Once a mapping unit was selected, sites within that unit were determined by a random number-grid scheme. This eliminated the natural bias to select sites away from the unit boundaries. On a few occasions, it was necessary to work at sites near, but not exactly at, the selected point to achieve landowner cooperation.

A morphological inspection of the soil was made at each prospective site to establish the accurate classification of the soil. If the morphological characteristics were within the range of values assigned to that particular soil series according to criteria of the Cooperative Soil Survey, the site was accepted as an experimental location. In this way, variation within soil series was measured, and not that within mapping units.

Methods used--The crust test method, as described in this Appendix was used for field measurement of hydraulic conductivity. It was selected because it can estimate K_v for saturated and unsaturated conditions, and it is accurate over the range of ψ_m where most moisture flow occurs.

Weighted least-squares, non-linear regression aided in fitting curves to the K data. Conductivity data were found to be generally log-normally distributed, as has been reported by Nielsen et al. (1973) and Mason et al. (1957). Therefore, log transformations were performed on the data. Following the transformations, the mean log K($\overline{x}_L$), standard deviations (S_L) and coefficients of variation of log K were calculated for intervals of log ψ_m. Variation was presented graphically as one standard deviation taken about the regression curve. It can also be expressed as the coefficient of variation with some restrictions due to the log-transformations.

When transforming back to linear data, the antilog of $\overline{x}_L$ is the geometric mean (G) of measured K (Snedecor and Cochran, 1967). The antilog of standard deviation of the logs (Antilog S_L) then becomes a multiplier of G, such that the 16th and 84th percentiles of the log normal distribution of K correspond to G/Antilog S_L and G_L Antilog S_L, respectively.

Multivariate discriminant analysis (Discrim 1 program, Schlater and Learn, 1975) was used to compare soil series on the basis of their distributions of ψ_m and K about their center of mass. This helped determine which data sets could be validly compared using this statistical technique. The results of the analysis as well as visual inspection of the curves were used to arrange the soil series into groups of similar K characteristics. Eisenbeis and Avery (1972) offer a thorough description of multivariate analysis and its limitations.

Variation within soil series--The log of the hydraulic conductivity data gathered for the Plano series is presented in Figure B-30. Each point represents a (log ψ_m, log K) pair resulting from steady-state flow through an artificial crust.

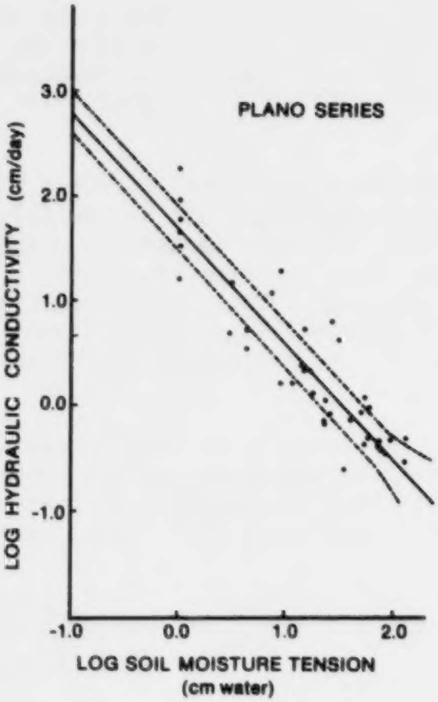


Figure B-30. Hydraulic conductivity as a function of soil moisture tension for the Plano series (Baker, 1976a).

To describe the relationship between K and moisture potential, it was necessary to fit a line to the data set. Linear regression yielded a good fit on log transformed data as shown in Figure B-30. The interval enclosed by one standard deviation is also presented. The mathematical model that best described this relationship was in the form of a power function.

$$K(\psi_m) = c\ \psi_m^{\ p_1}$$

where K = the hydraulic conductivity at potential ψ_m,

c = a constant, such that log c = log K_{sat} - p_1,

ψ_m = an absolute value of soil moisture potential,

p_1 = a power whose value is negative.

If the logs of both sides are taken the equation becomes log $K(\psi_m)$ = log c + b_1 log ψ_m or $y = c' + p_1 x$, where p_1 is the slope of the line and c' is the y intercept of the line at log $\psi_m = 0.0$. Because the transformed data can be described by a simple line many standard mathematical procedures can be applied to the data, such as the construction of prediction intervals (Baker and Bouma, 1976b).

This function provides satisfactory fits for the data of four other soil series: Hochheim, Magnor, Ontonogon and Withee (Figures B-31, B-32, B-33 and B-34, respectively). The values of parameters c and p_1 are indicated in the figures. The variation about the regression line is not greatly different among the series, as indicated. For these series variation was fairly large. All five of these soils were well-aggregated except the Hochheim, which has a weak blocky structure. The morphological properties of the soils can be compared in Table B-4.

Other soil series could not be satisfactorily described by the above power equation, because they were non-linear. Among these was the Boone sand, the log-transformed data of which is shown in Figure B-35. The indicated curve can be described by adding an exponential term to the power equation.

$$K(\psi_m) = c\ \psi_m^{\ p_1} + \frac{K_{sat}}{\text{Antilog}(\psi_m/\psi_c)^{p_2}}$$

where $K(\psi_m)$, c, ψ_m and p_1 are as described for the previous equation,

K_{sat} = the mean saturated K of the soil,

ψ_m = a critical tension, being the potential at which $K(\psi_c) = 0.1\ K_{sat}$,

p_1 = a power whose magnitude correlates roughly with the homogeneity of pore sizes contributing to flow for this range of moisture potentials ($p_1 > 1.0$).

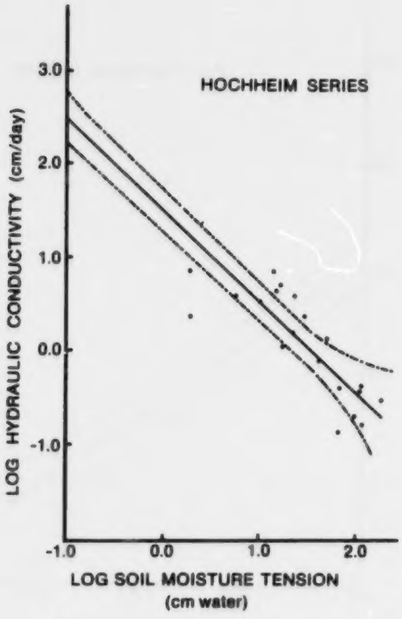


Figure B-31. Hydraulic conductivity as a function of soil moisture tension for the Hochheim series (Baker, 1976a).

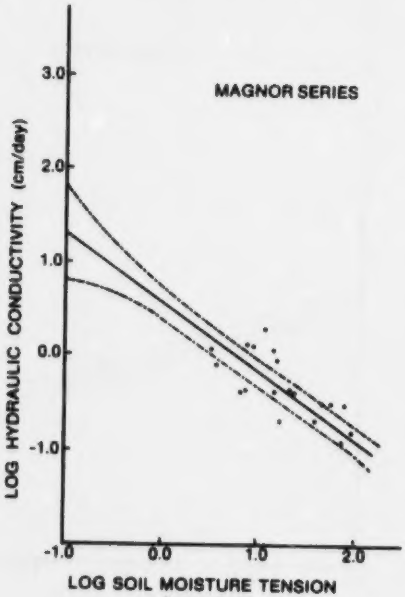


Figure B-32. Hydraulic conductivity as a function of soil moisture tension for the Magnor series (Baker, 1976a).

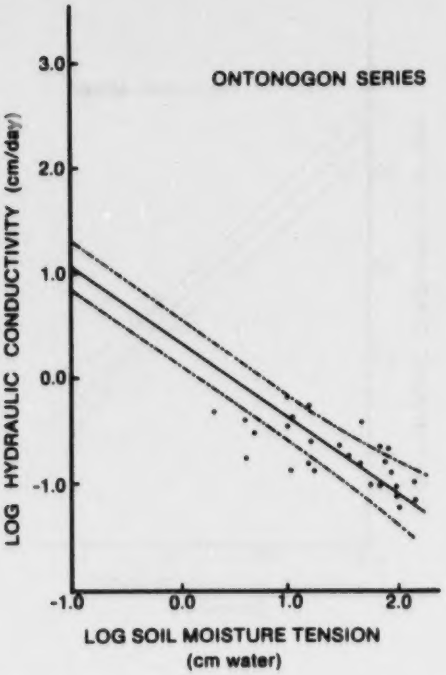


Figure B-33. Hydraulic conductivity as a function of soil moisture tension for the Ontonogon series (Baker, 1976a).

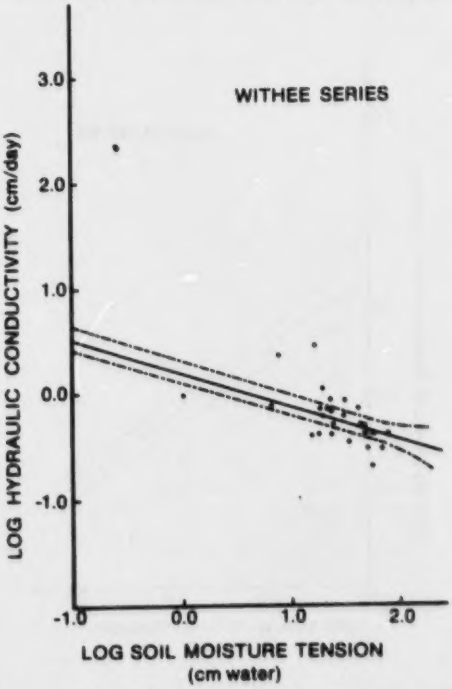


Figure B-34. Hydraulic conductivity as a function of soil moisture tension for the Withee series (Baker, 1976a).

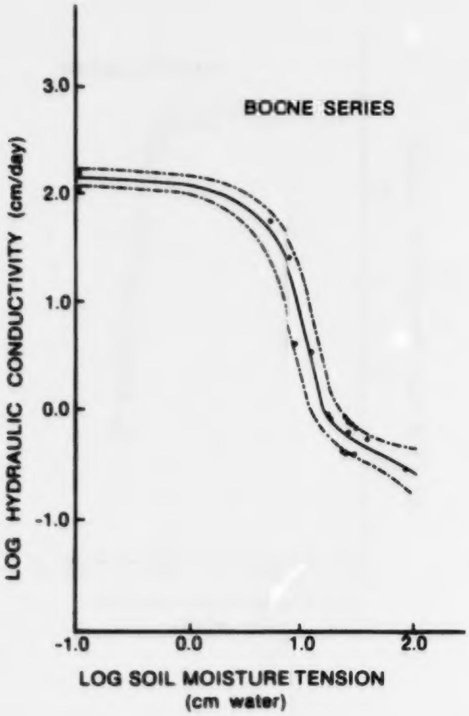


Figure B-35. Hydraulic conductivity as a function of soil moisture tension of the Boone series (Baker, 1976a).

In the above equation, (ψ_m) is the tension which occurs when K equals one tenth of the value of K_{sat}, because this relationship is inherent in the behavior of an exponential. It corresponds with x = 1.0 on the abscissa of a unit exponential graph (see Figure B-30 to B-38). As such, ψ_m is simply a scaling factor, which can be visually approximated from the untransformed data set. Good fits can be made using whole numbers for ψ_m.

Several expressions have been suggested (Raats and Gardner, 1971) which almost fit the Boone data and similar series. These expressions generally do not describe the data well for the potential range of 15 to 100 cm water, except in coarse materials. For prediction of a soil's response to specific wastewater management, K data must be accurately described in this range (Bouma, 1975; Baker and Bouma, 1976a). The above equation has proven somewhat more useful for this purpose.

Three other soil series (besides the Boone) which were well described by the above equation were the Plainfield, Morley and Batavia. Their log-transformed data sets and regressed curves are found in Figures B-36, B-37 and B-38, respectively. In each figure the equation for the curve is given.

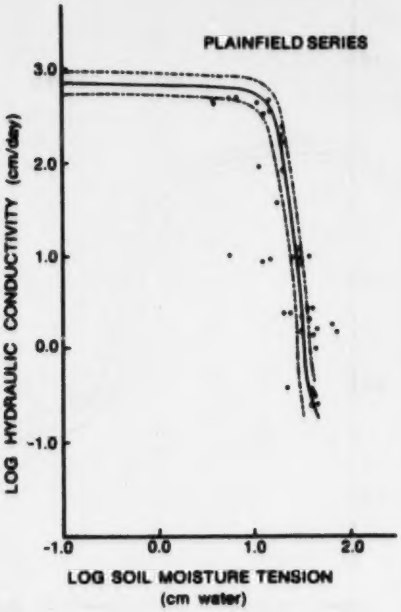


Figure B-36. Hydraulic conductivity as a function of soil moisture tension of the Plainfield series (Baker, 1976a).

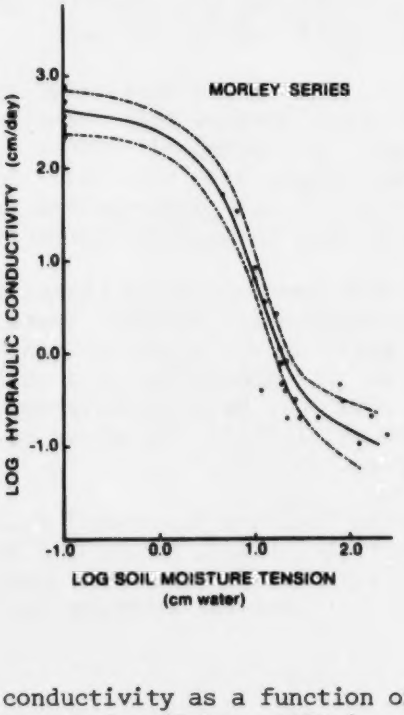


Figure B-37. Hydraulic conductivity as a function of soil moisture tension of the Morley series (Baker, 1976a).

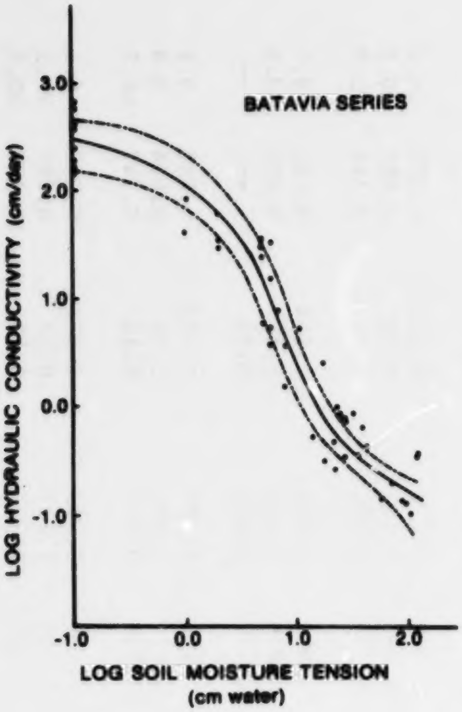


Figure B-38. Hydraulic conductivity as a function of soil moisture tension of the Batavia series (Baker, 1976a).

The amount of variation of the data about the regression curve is large in several of the soil series. The Plainfield series, for example, includes some rather large deviations from the curve. Calculation of logarithmic means ($\overline{x}$), standard deviations (S_L) and geometric means G and other statistics (Rogowski, 1972) are presented in Table B-5.

Because variations within some soil series are rather large, accurate prediction of expected conductivity values may not be feasible. Ninety-five percent prediction intervals constructed for the Plano and Batavia series (Baker and Bouma, 1976b) enclose K values that differ by nearly an order of magnitude. However, knowledge of the variability of K for a given series does permit, to a certain extent, estimation of the minimum expected value for a particular moisture potential for a desired confidence level. Siting of a soil absorption field is one use that would benefit by this information.

Not all soil series were as variable as the Plainfield series. The Hochheim series exhibits considerably lower variation about the regression curve. This would allow more accurate estimation of the range of values of K for a given moisture potential.

Variation between soil series--Multivariate discriminant analysis was used to help determine which of the soil series could be compared to one

TABLE B-5. SOIL STATISTICS FROM THE HYDRAULIC CONDUCTIVITY VARIABILITY STUDIES (Baker, 1976a).

Soil Series	n	Soil moisture potential (ψ_m) (cm)	Geometric mean G	Antilog S_L	$\bar{x}_L$	S_L (soil and method)	Average correction for method variation	S_L (soil only)	CV (%)
Withee	5	0	3.94	1.31	0.596	0.116	0.032	0.084	14.1
	4	15	0.49	1.30	-0.308	0.115	0.10	0.015	4.9
	4	50	0.42	1.11	-0.378	0.046	0.10	--	--
Ontonogon	6	0	15.87	1.63	1.200	0.211	0.032	0.179	14.9
	4	15	0.32	1.42	-0.495	0.152	0.10	0.052	10.5
	4	60	0.176	1.84	-0.755	0.265	0.10	0.165	21.9
Magnor	5	0	28.37	2.910	1.453	0.464	0.032	0.432	29.7
	4	15	0.52	1.51	-0.244	0.178	0.10	0.078	32.0
	3	60	0.28	1.264	-0.590	0.102	0.10	--	--
Hochheim	7	0	292.3	77.71	2.466	0.277	0.032	0.245	9.9
	4	15	4.77	1.34	0.678	0.126	0.10	0.026	3.8
	4	60	0.30	2.223	-0.529	0.347	0.10	0.247	46.7
Plano	11	0	644.3	1.547	2.809	0.189	0.032	0.157	5.6
	4	15	2.75	1.575	0.439	0.197	0.10	0.097	22.1
	4	60	0.376	1.589	-0.425	0.201	0.10	0.101	23.8
Batavia	10	0	271.22	1.75	2.430	0.244	0.032	0.212	8.7
	4	10	2.48	1.77	-0.394	0.248	0.10	0.148	37.6
	4	40	0.494	1.51	-0.306	0.179	0.10	0.079	25.8
Morley	5	0	380.65	1.69	2.581	0.228	0.032	0.196	7.6
	4	15	2.21	1.74	0.344	0.240	0.10	0.140	40.7
	4	35	0.29	1.68	-0.542	0.226	0.10	0.126	23.2

-continued-

TABLE B-5. (continued)

Soil Series	n	Soil moisture potential (ψ_m) (cm)	Geometric mean G	Antilog S_L	$\bar{x}_L$	S_L (soil and method)	Average correction for method variation	S_L (soil only)	CV (%)
Boone	3	0	131.65	1.19	2.119	0.075	0.009	0.066	3.1
	3	25	0.82	1.26	-0.087	0.100	0.080	0.020	23.0
	4	40	0.59	1.31	-0.253	0.116	0.080	0.036	14.2
Plainfield	6	0	734.15	1.32	2.866	0.121	0.009	0.112	3.9
	4	21	123.69	2.196	2.092	0.342	0.080	0.333	15.9
	4	35	4.561	2.402	0.659	0.381	0.080	0.301	45.7

another. This was accomplished by simultaneous tests of the similarity of distribution centers of mass and of the similarity of distribution variances (dispersions) about that center. This would in theory indicate which series can be placed together into groups of similar hydraulic conductivity properties. However, in practice, these results must be considered only estimates, because in real situations data sets rarely meet the statistical characteristics required for the test to operate correctly (Pavlik and Hole, 1977; Eisenbeis and Avery, 1972).

The Morley, Batavia and Plano series exhibited very similar hydraulic properties and could be grouped together. Figure B-39 illustrates their regression curves. Visually and statistically these series are very closely related. A comparison of the general morphological information (i.e., textural and structural data) for each of these series also show minor differences. These series coalesce to form a common conductivity group, that will be temporarily labeled here the "Batavia" group.

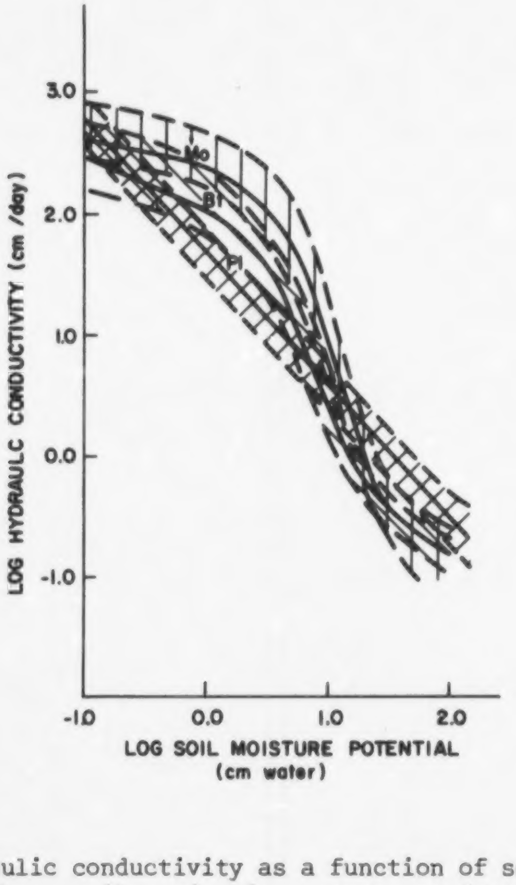


Figure B-39. Hydraulic conductivity as a function of soil moisture tension for three soil series in a common conductivity group (Baker, 1976a).

Another conductivity group is formed by the Magnor and Ontonogon series. These regression curves are presented in Figure B-40 for visual inspection. Morphological properties are similar. This conductivity group will be temporarily named the "Magnor" group.

The data sets for the four remaining soil series, Plainfield, Boone, Hochheim and Withee, were distinct and could not be placed together into conductivity gmoups. Comparison of their regression curves in Figure B-41 supports this conclusion.

In Figure B-41, the regression curves for distinct conductivity data sets are shown. Only one curve is shown for each of the two above discussed conductivity groups, the Batavia and the Magnor, for their respective groups. It can be seen that the various curves are easily distinguishable. As saturated conditions are approached great disparities become apparent. In this range of soil moisture potentials, soil macropores appear to dominate moisture flow. These include structural cracks and biopores such as root and worm channels. As the soil becomes drier, K decreases markedly, as the effects of the larger pores decrease. This transition occurs gradually until at a potential of about 30 cm, most of the curves flatten and appear to approach some low K value asymptotically. In this region of the graph, several of the curves are very similar, becoming indistinguishable from one another. This suggests that exact classification of soil series for this range of soil moisture conditions may not be necessary, since K values differ little among series.

Effective planning and management for many land uses require the accurate prediction of expected hydraulic conductivity values at a given site. One way to provide this information is through the construction of prediction intervals to include the expected value of K at any future site. Figure B-42 is such a prediction interval for the Plano series at the 95 percent confidence level. The range of values enclosed in an interval of such high statistical assurance is relatively large. However, planning and management of a soil use such as liquid waste disposal requires prediction of the minimum expected K at a site, not the range of expected values (Baker, 1976b). Therefore, the lower prediction limit (line 2 in Figure B-42) can be used to project whether the required conductivity can be expected for a given soil series and value.

Summary and conclusions--Hydraulic conductivity data sets for nine soil series exhibited relatively high variability. An empirical function was regressed through each data set achieving good fits in all cases. Multivariate discriminant analysis was used to help determine which soil series had similar hydraulic properties and could therefore be grouped together. Five of the soil series could be formed into two conductivity groups.

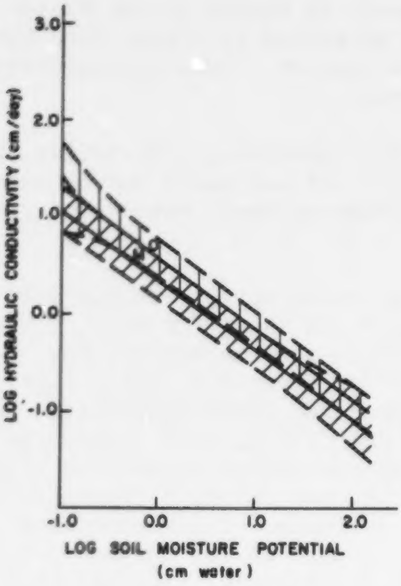


Figure B-40. Hydraulic conductivity as a function of soil moisture tension for two series in a common conductivity group (Baker, 1976a).

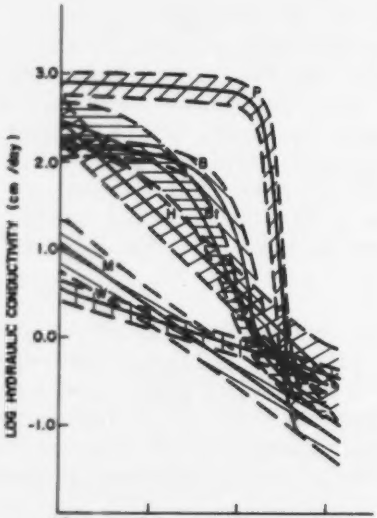


Figure B-41. Hydraulic conductivity as a function of soil moisture tension for soil series of dissimilar conductivity (Baker, 1976a).

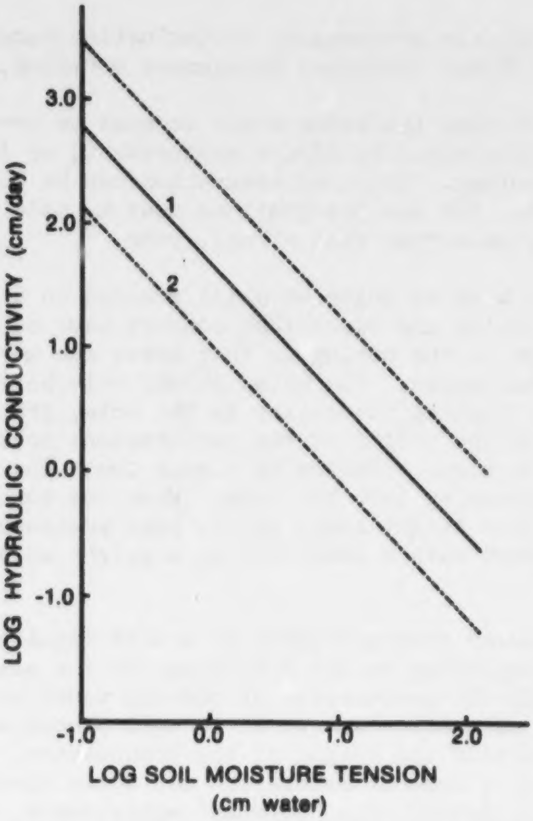


Figure B-42. Hydraulic conductivity as a function of soil moisture tension with 95 percent prediction limits (Baker, 1976a).

Other Factors

Depth of Soil to the Water Table or Bedrock--

Adequate purification of effluent may take as much as three feet of unsaturated soil. This distance has been shown to remove all constituents except NO -N if loading rates are not too high and short circuiting is avoided (see Appendix C). Not only does soil saturation limit the purification capability, it may also reduce the hydraulic potential difference. Also, under these conditions a more intense clogging layer may form restricting flow even more.

Bedrock may affect flow in two extreme ways. Non-porous bedrocks such as granites or shales may stop flow, causing the water to either flow laterally or back up. Creviced bedrock, such as that found in limestone formations may allow wastewaters to pass very rapidly through the cracks to the groundwater with little or no treatment. Septic tank soil absorption systems in Door County Wisconsin, once believed to be working very well, have been found to be the cause of considerable groundwater contamination because of the shallow creviced limestone (Water Resources Management Workshop, 1973).

Water table--Whether the water table is real or perched, the zone of saturation may be determined by direct measurement, or it can be estimated based on soil morphology. Zones of saturation can be determined by direct observation of wells, but the observations must be made during the wettest period of a normal, or wetter than normal, year.

Holes prepared with an auger or drill mounted on a truck should be cased with tubing. PVC tubing and electrical conduit have been used as casing. Holes should be made in the casing so that water can move easily in and out but other material cannot enter. The holes should only be located in the horizon of interest. After placing the casing in the hole, gravel may be placed around the casing to the height of the perforations to keep the soil on the sides of the hole in place. Bentonite cement above the gravel will keep surface water from running into the hole. When the zone saturates, water will run into the hole to a height equal to the head pressure of the layer. Measurements may be taken with a small rod or a weight on a string placed in the hole.

Proper groundwater interpretation of a site requires that wells be placed at various depths depending on the morphology of the soil. In a uniform coarse textured soil the groundwater of concern would be continuous from the top of the zone of saturation downward. A well placed within the zone of saturation would reflect the height of the groundwater. If a horizon of slow permeability exists, a zone of saturation may occur above it but not necessarily below it. To detect this "perched" water table, a well should be placed only as deep as the top of the less permeable horizon. If the well is placed below the horizon, water may not enter the well or it may enter from above the restricting layer and exit below. Because direct monitoring of saturation is time consuming and weather dependent, interpretation of soil morphology is often used to estimate seasonal soil saturation.

Soil minerals, organic matter, and compounds of iron and manganese contribute to soil color. If these materials are uniformly mixed, the soil will have a uniform color. Colors are not uniform in many soils, but are mottled in patches of reds, oranges, browns and grays. Soil mottles are defined as "spots of color" (Soil Survey Staff, 1951) or "spots of contrasting color" (Soil Survey Staff, 1975) and may result from segregation of soil components by transport and precipitation during periods of changing moisture conditions. Soil mottling or lack of mottling can usually be used as an indication of the soil moisture conditions throughout the year. Drainage classes (Soil Survey Staff, 1951) and soil moisture regimes (Soil Survey Staff, 1975) are defined using soil mottling and offer ways to classify moisture conditions.

Mottles form during soil saturation because of the chemical environment that develops. When soils are saturated or nearly saturated, dissolved oxygen in the soil solution can be removed by microorganisms if the conditions are right. As O is removed, reducing conditions develop, and forms of Fe and Mn become more soluble. Spots produced where Fe and Mn have been reduced or removed are gray and are generally referred to as mottles of chroma 2 or less (Soil Survey Staff, 1975).

The more soluble reduced forms of Fe and Mn are free to go with moving water until points are reached where oxidizing conditions exist. At this point less soluble Fe and Mn compounds form. Just where they form in the soil will depend on the configuration of the soil system. In the field, oxidized forms of Fe appear red and those of Mn black. The oxidized Fe and Mn compounds are stable and will persist in the soil even when the soil is dry.

Occurrence of low chroma mottling (chroma < 2) can be used in most cases to predict the existence of significant periods of saturation. Correlation of duration of annual periods of saturation with morphological characteristics and low chroma mottles have been made (Daniels and Gamble, 1971; Simonson and Boersma, 1972; Veneman et al., 1976) but just how long it takes to form mottles is not known (Soil Survey Staff, 1975). Laboratory experiments have shown that gleying (Daniels et al., 1973) and mottling (Vepraskas and Bouma, 1976) can develop in very short time periods under proper conditions. It is of primary importance to determine the degree and duration of saturation required to produce and maintain a given morphological expression of soil mottling so that this can be interpreted in terms of estimates of degree and duration of saturation.

Though soil morphological characteristics, particularly soil mottling, have been used successfully in many cases to estimate annual soil moisture conditions (especially periods of saturation) there are times when it has not worked. Errors can result because of a soil chemical environment not conducive to generation of mottles or because mottles formed by a genetic process not associated with the present moisture regime. Studies have shown that correlations of the extent of saturation with mottling can be good (Daniels et al., 1971; Simonson and Boersma, 1972), and that detailed morphological description of mottles can be used to define soil moisture conditions more precisely (Veneman et al., 1976; Vepraskas et al., 1974).

In Wisconsin, detailed studies of soil moisture and mottling have been made around Madison. In these studies three broad categories of moisture regimes and associated morphological features were distinguished (Veneman et al., 1976).

Category 1: These horizons were saturated for only short periods of time and generally less than one day. Mottles with chromas of two or less (grayish) did not occur. Processes of reduction in this horizon were strong enough to reduce Mn and form ped mangans (ped coatings high in Mn) and Mn nodules.

Category 2: These soil horizons show saturation for periods of several days at a time. They are nearly saturated for several days at a time with only the largest pores drained. Mottling dominated by chromas of two inside peds and iron cutans (ped coatings high in iron called neoferrans or ferrans) along larger pores with a few manganese mottles is associated with this moisture condition. A drier but similar condition was discussed on a different soil (Vepraskas et al., 1974).

Category 3: The horizons in Category 3 were saturated for a period of several months. All soil pores would be water-filled. Mottling dominated by low chromas of one (gray) inside peds, ped and channels neoalbans and virtual lack of manganese mottles are present when saturation occurs for several months. "Neoalbans" was suggested as a name for the gleyed zone around soil pores produced as water entering a ped carried the surface iron away.

Though detailed morphological descriptions may be helpful to distinguish detailed moisture conditions, they have not been used in routine on-site evaluation. Instead soils with mottling have been considered saturated or nearly saturated, and, if at a shallow depth, the site deemed unsuited. Though direct groundwater observation may be the best method of establishing zones of saturation, interpretation of soil mottling is usually adequate.

Bedrock--Determining the depth to bedrock in many cases is very easy but in others the weathering process has resulted in a zone of questionable character. The rock can become very soft or break down into gravel sized or smaller pieces.

The soil conservation service defines lithic and paralithic contacts for distinguishing bedrock based on crack spacing, hardness and slaking character (Soil Survey Staff, 1975). Health codes have used the criteria of percent hardrock and the ability to excavate with ordinary construction equipment.

Determining the percent hardrock may depend on what is considered hard. This has not been well-defined. In many cases the hardrock may be distinguishable, but the percentage difficult to determine. A piece of wire mesh placed over the profile face may be used to more accurately determine the percent by counting the wire intersections that occur over hardrock. This has been used successfully in areas of creviced limestone bedrock.

Excavation with ordinary construction equipment will indicate the possibility of installation of a subsurface system and may indicate hardness of the material. Soil material containing considerable rock and some weathered bedrocks may be excavated but may be unsuited for a soil absorption bed. Consideration of the soil material should be used as a deciding criteria.

Slope--

There is concern that effluent may surface downslope from a soil absorption system. This may occur if a flow restricting subsurface horizon is present and the effluent passes above it to a seep area. In the absence of a restricting horizon, flow is dominantly downward and should not create a problem (Bouma, 1977). Steep slopes may place severe restrictions on the use of equipment for construction.

Land slope can be expressed and determined in many ways. Slopes have been expressed as:

1. Percent of Grade - The feet in vertical rise or fall in 100 feet of horizontal distance.
2. Slope - The ratio of vertical rise or fall to horizontal distance.
3. Degrees and Minutes - The angle of slope measured horizontal.
4. Topographic Arc - The feet of vertical rise or fall in 66 feet of horizontal distance.

Land slopes are usually determined by measuring the slope of a line parallel to the ground with an Abney Level at some fixed height. The value of the slope as percent or in degrees is read directly from te he instrument. A hand level may be used on a horizontal line of site. If a survey of the area is being made slopes may be determined from this information.

Landscape Position--

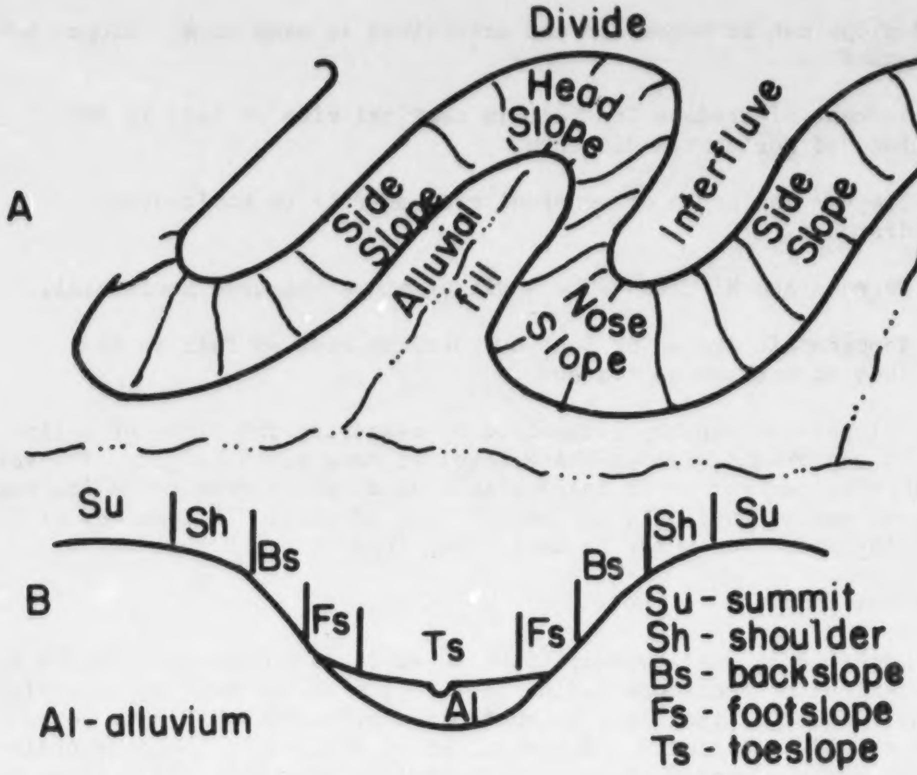
The location in the landscape such as at the shoulder (see Figure B-4), or on the backslope or footslope influences the surface as well as subsurface water (the landscape itself may be controlled by bedrock and thus certain positions should be avoided). The shoulder of a slope is the best drained. This is generally a convex slope. The backslope generally has surface water running over it, but it generally continues on downslope. Depending on the underlying stratigraphy at some places on the backslope, a groundwater seep may be found. These spots can usually be seen as depressions on the slope.

The foot of the slope is where the water running off the backslope slows down. Generally these areas are wetter than other segments of the slope. These regions are concave in nature. Floodplains or bottoms also have excessive surface water. Though any of these areas may prove to be suitable for some type of on-site waste disposal, certain landscape positions are generally better than others.

Position in the landscape can be determined visually or from a detailed survey. aFactors that could be considered to give some indication of the surface and soil water conditions should be recorded. These might include the position, as illustrated in Figure B-43, and the shape of the landscape element such as concave or convex.

Management History--

Man uses the soil for many other purposes besides on-site waste treatment. Some of these activities do not alter the properties of the natural soil appreciably, while others do. Heavy machinery or repeated trips over an area with light machinery can compact the soil, increasing its bulk density and decreasing its permeability.



Divide
A
Head Slope
Side Slope
Alluvial fill
Interfluve
Side Slope
Nose Slope
B
Su
Sh
Bs
Fs
Ts
Al
Fs
Bs
Sh
Su
Al - alluvium
Su - summit
Sh - shoulder
Bs - backslope
Fs - footslope
Ts - toeslope

Figure B-43. Terminology of hillslopes according to Ruhe (1969).

Cutting and filling is frequently done to change the nature of a landscape. This operation leaves an area with subsurface material exposed, which might be quite different from what was once at the surface, and a filled area of mixed material. The properties of the fill, particularly in fine-textured materials, will be different than before it was moved and will change with time.

Past use of an area is sometimes difficult to determine. Factors noted during a site evaluation that might indicate that the site has been significantly altered include the lack of surface soil horizons, sharp unnatural changes in slope, soil material that appears to be a mixture of different soil horizons or has artifacts mixed in with the material and layers that appear compressed or compacted. Before testing, it should be determined if the material has stabilized and properties will not change further with time. Also, it should be established if the material is uniform. If these factors are positive, normal site evaluation procedures can be performed.

DETERMINATION OF SITE SUITABILITY BASED ON SOIL INFORMATION

Consideration of methods to utilize soil variability information for predicting, with a certain probability, whether the hydraulic conductivity at a given site will allow a liquid waste disposal system to operate properly is necessary. Three possibilities have been discussed previously (Bouma, 1974; Baker and Bouma, 1976a), and will be summarized briefly here.

The first of these is direct on-site testing of permeability at any new site. This is already required in many health codes (USPHS, 1967) as a means of evaluating a site. Use of the crust test would yield a K curve for the soil. Sizing of the seepage system would then be based on the expected loading rate and on the capacity of the soil to accept liquid. The latter characteristic can be determined from observed tension corresponding to a steady-state flow rate that occurs once a stable clogging layer is established. Thus, this steady state conductivity can be found from the K curve. The soil moisture tension under the bed can be measured and the value of the conductivity extrapolated to other sites in similar soils.

A second possibility would require a knowledge of the mean conductivity characteristic of soils at the soil series level and information on the variability of K. To use these data, on-site determinations in specific soil series would be necessary, and the data for the series would be used for evaluation. The third scheme would eliminate most on-site field determinations by the use of detailed soil maps already available in many areas from the Cooperative Soil Survey, U.S. Department of Agriculture. All of these possibilities have exceptions, reverting the process back to the previous scheme.

Figure B-44 is a schematic breakdown of these systems of site evaluation, indicating the major steps in each process. This figure is based on the application of the conventional septic tank-seepage field system. It also assumes that other limiting conditions are screened out separately, and that the conventional percolation test is the means of measuring permeability. However, the scheme is also valid for crust test data, and this aspect is most interesting. Using more advanced techniques to measure K could increase the accuracy of any of these systems. This could also allow quantitative decisions to be made such that alternate methods of disposal would be possible, and would allow a more accurate basis for sizing of the field (Baker, 1976b). The diagram is self-explanatory, but a few major points are stressed below.

System I--

System I is the decision-making process now used in Wisconsin and several other states. Based on percolation test data, as well as other environmental factors, construction is either approved or denied. Each site is tested separately. If it does not meet requirements, the owner cannot construct a conventional system. If it passes, the field is sized according to percolation test data following empirical procedures which do not consider soil permeability alone (USPHS, 1967). Use of the crust test would greatly improve the sizing procedure.

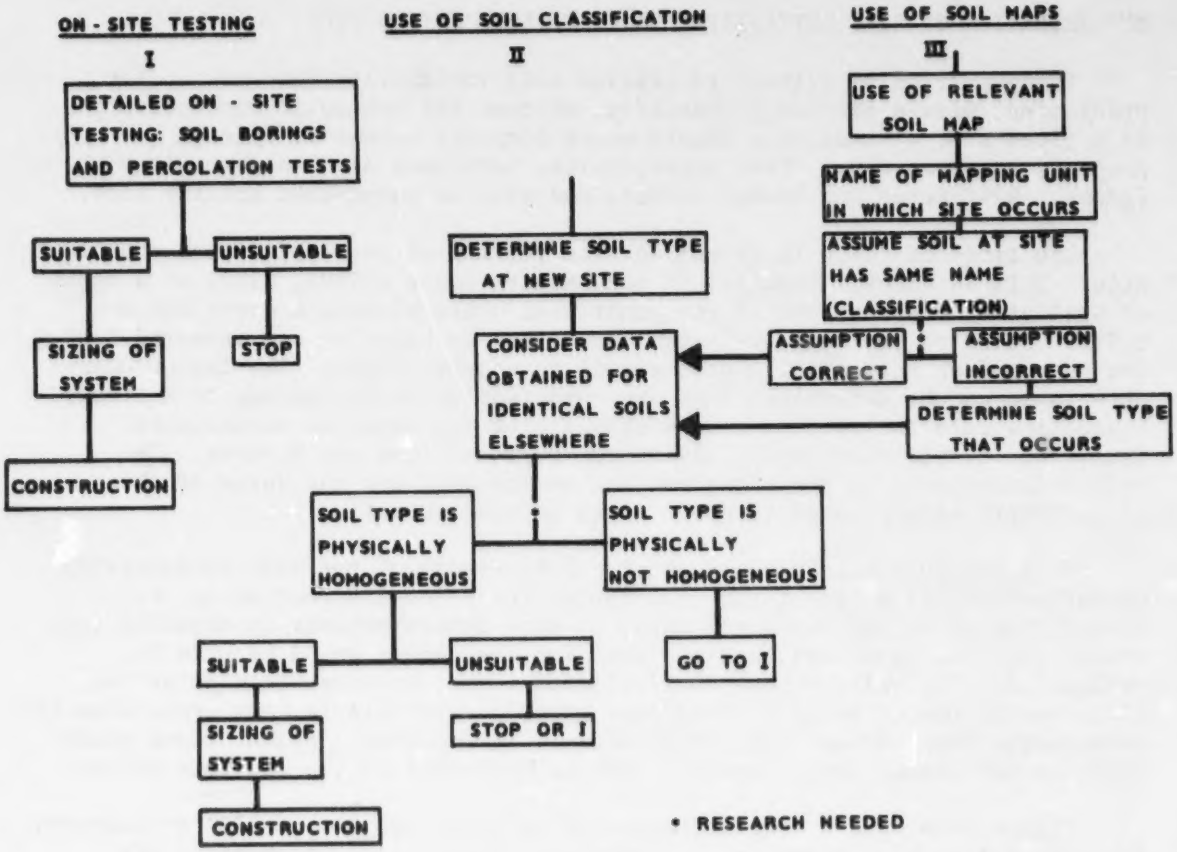

ON - SITE TESTING
I
DETAILED ON - SITE TESTING: SOIL BORINGS AND PERCOLATION TESTS
SUITABLE
UNSUITABLE
SIZING OF SYSTEM
STOP
CONSTRUCTION
USE OF SOIL CLASSIFICATION
II
DETERMINE SOIL TYPE AT NEW SITE
CONSIDER DATA OBTAINED FOR IDENTICAL SOILS ELSEWHERE
SOIL TYPE IS PHYSICALLY HOMOGENEOUS
SOIL TYPE IS PHYSICALLY NOT HOMOGENEOUS
SUITABLE
UNSUITABLE
SIZING OF SYSTEM
STOP OR I
CONSTRUCTION
GO TO I
USE OF SOIL MAPS
III
USE OF RELEVANT SOIL MAP
NAME OF MAPPING UNIT IN WHICH SITE OCCURS
ASSUME SOIL AT SITE HAS SAME NAME (CLASSIFICATION)
ASSUMPTION CORRECT
ASSUMPTION INCORRECT
DETERMINE SOIL TYPE THAT OCCURS
• RESEARCH NEEDED

Figure B-44. Measurement of soil hydraulic conductivity and site selection.

System II--

In System II, based on accurate measurements from the crust test or other methods, the hydraulic conductivity characteristics of major soil groupings are determined including variability data. This could follow much the same procedure as was described for the variability experiments presented by Baker (1976a). Some soil series would coalesce into conductivity groupings whose mean characteristics would be similar, so the number of these divisions would not be too large. A field determination as to which soil series occurs at the site would then lead to a realistic estimate of the conductivity characteristics of the site in question. Sizing of the drainfield would proceed based on the minimum expected hydraulic conductivity (after clogging). The field percolation test could not be used effectively for this purpose. Figure B-45 illustrates the range of possibilities that can arise under this scheme. If soil series at a given site has the K characteristic shown in

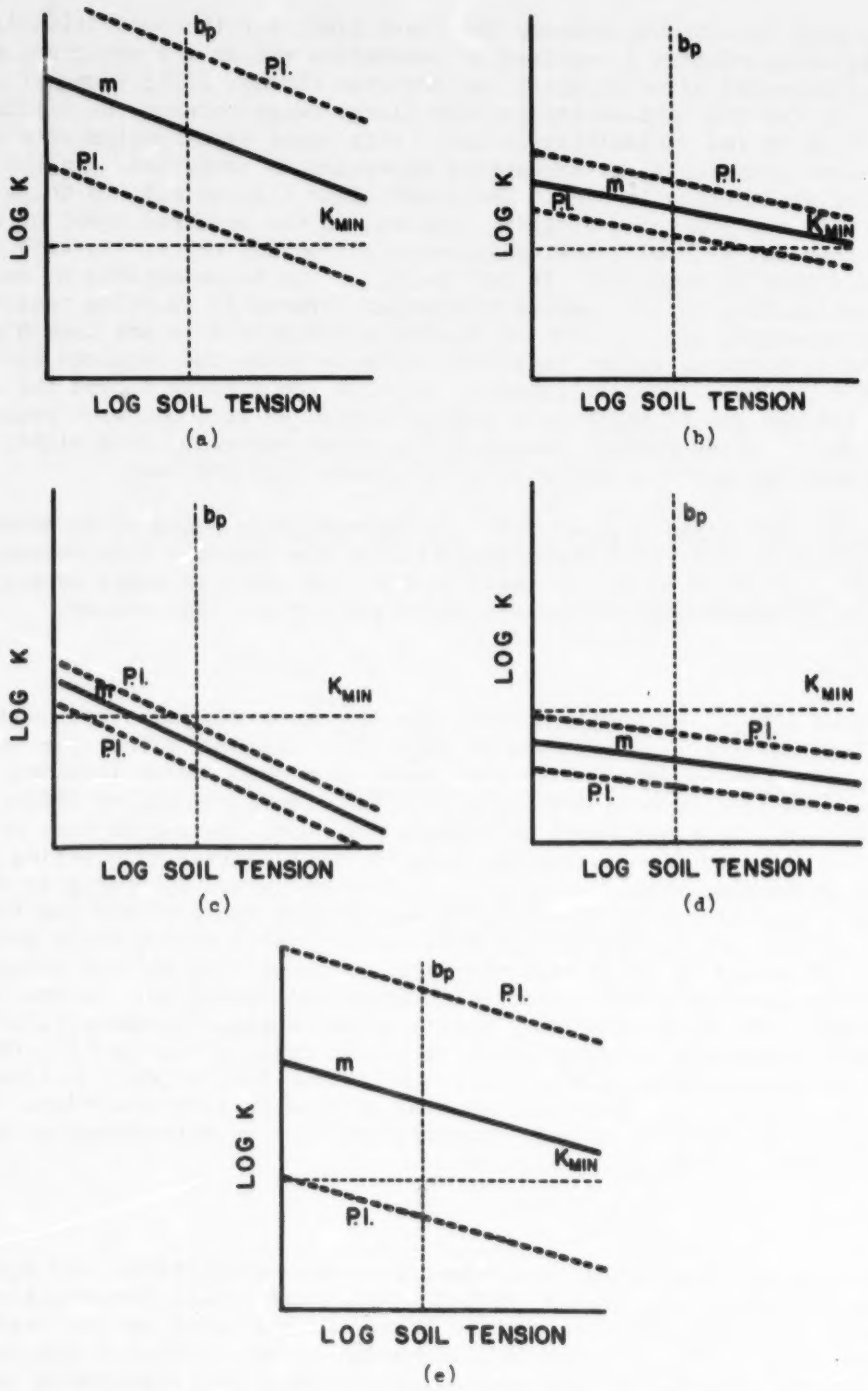


Figure B-45. Measurement of soil hydraulic conductivity and site selection for liquid waste disposal.

(a), the soil is suitable because the lower limit for the prediction interval (P.I.) is above minimum K required at saturation and at the equilibrium tension (bp) expected after clogging has occurred (Bouma, 1975; Bouma et al., 1975a). In (a) the variability is high (large range between the limits for P.I.), while in (b) variability is low. Both cases pass requirements because the required probability of successful operation is satisfied. In the case of (c) the variability is high. The lower limit (LL) of P.I. is below the minimum K at the predicted clogging tension, so the required level of confidence that the site will meet requirements does not exist. On-site testing of K would then be required. In (d) the P.I. is below minimum K, even though variability is low (narrow prediction interval). On-site testing would be required, but the odds of finding a suitable site are less because the probable range of values is almost entirely below the required minimum K In (e) the entire prediction interval is below the minimum K, and the site is rejected because of the high probability that it will not meet requirements. There is the outside chance that a given parcel of land might, with on-site testing, yield a usable site, but these odds are low.

In such a scheme, a great deal of information is required to establish the hydraulic conductivity characteristics of the possible soil series or groupings. In areas of highly variable soil groupings it might be more practical to simply perform on-site tests and not use this system.

System III--

System III is an extension of System II. Here soil maps are used for identification of the soil series or grouping at a given site. For this purpose, it is important to know the amount of mapping error involved. In some areas mapping is more difficult than in others, leading to differing map reliabilities. In cases where experience has shown the map is very reliable, a decision is based directly on the data for that series, eliminating the need for an on-site visit. This site is then evaluated according to the advanced stages of System II. Mapping reliability could be too low to allow this procedure, and an on-site identification of soil series would then be needed. It should be noted that the mapping reliability and the level of K probability for a site will compound. For certain soils this scheme can be very useful. In the central sand plains of Wisconsin, for example, the Plainfield series can be mapped with very high reliability, and its range of conductivities is almost always above the minimum K required. In this case, use of the soil map can save a great deal of time in site selection. Scheme III then relies not only on soil mapping, but also on determination of K data and variability of the soils involved.

Summary--

Soil suitability can be determined in a more quantitative and systematic way than described here (Baker, 1976a). To do this, soil permeability and wastewater system performance must be known, so that they can be treated as probability terms. Since this information is only available for very limited circumstances, the implementation of such a decision-making scheme is not possible at this time. However, the potential improvement in accuracy of prediction is well worth pursuing.

PART II

WASTEWATER SOIL ABSORPTION SYSTEMS

On-site subsurface soil disposal of septic tank effluent is the most common means of domestic liquid waste treatment in unsewered areas. In 1970, approximately 16.6 million housing units or approximately 25 percent of all housing units in the United States disposed of their wastewaters via septic tank-soil absorption systems (Cooper and Rezek, 1977). The use of these systems is growing at a rate of about one-half million new systems per year (Patterson, Minear and Nedved, 1971). This rate is increasing due to an emerging trend of population movement to rural areas (Beale and Fuguitt, 1975). Because of poor design, construction or maintenance practices, however, a large percentage of these sytems are failing to provide adequate treatment and disposal of the wastewater.

Failure usually manifests itself by seepage of septic tank effluent over the ground surface or by sewage back-ups in the household plumbing due to a severely clogged soil absorption field. Since the system is usually near the dwelling, the poorly treated waste can become a nuisance, as well as a health hazard. A more serious type of failure, however, occurs when there is insufficient or unsuitable soil below the absorption field to properly purify the wastewater before it reaches the groundwater. Contamination of nearby wells by bacteria, viruses and chemical pollutants can result. This type of failure can go unnoticed until illnesses or epidemics occur.

Increased emphasis on environmental quality and public health calls for satisfactory treatment and disposal of domestic liquid wastes for all homes in unsewered areas. This concern is expressed by regulatory codes which limit the use of on-site disposal systems to soils which are often arbitrarily defined as being "suitable" for the conventional septic tank system. Because the pressure for development of unsewered areas is great, these codes are difficult to enforce since it is estimated that only about 32% of the total land area of the United States meet these accepted soil criteria (Wenk, 1971). However, recent advances in soil physics, soil chemistry, microbiology and engineering are leading to improvements in the conventional design as well as alternative designs of on-site systems which overcome the limitations imposed by many "unsuitable" soils.

MAINTAINING THE INFILTRATIVE CAPACITY OF THE SOIL

Soil Clogging

Proper performance of an on-site wastewater system utilizing soil absorption for ultimate disposal of the liquid depends upon the ability of the soil to completely absorb and purify all the wastewater produced. Initially, the soil may have a high infiltration rate and is able to absorb a greater quantity of liquid than applied. However, with continued application of wastewater, a clogging mat usually develops at the infiltrative surface. This creates a barrier to flow, restricting the rate of infiltration. Clogging, per se, is not synonomous with failure because flow through the mat will continue, albeit at a reduced rate. In fact, some clogging is beneficial to enhance purification. Therefore, proper design requires that the wastewater loading never be allowed to exceed the infiltration rate of the clogged soil or that the extent of clogging be controlled to maintain the desired infiltration rate.

Soil clogging is a complex phenomenon that is the result of many mechanisms, often acting simultaneously. They can usually be grouped into physical, chemical and biological processes. Not all mechanisms are equally significant, however. In their review of the literature, McGauhey and Krone (1967) concluded that biological agents and their activities are the most important cause of soil clogging. This contention is supported by several investigators. Allison (1947) experimented with sterile and unsterile loam and sandy loam soil columns, ponded with sterile and non-sterile water. Only the sterile columns ponded with the sterile water did not show the characteristic decline in infiltrative capacity usually seen in continuously inundated soil. Bacterial populations in the soils from the various columns used provided further evidence that the clogging was caused by microbes.

McCalla (1945, 1946, 1950) also did work on the effects of microorganisms on the rate of percolation of water through soils. In one study, three sets of columns containing a sand loam soil were prepared (McCalla, 1950). One set received distilled water only, another was covered with a cotton gin waste mulch before distilled water was applied, and the third had mercuric chloride added to the water to act as a disinfectant. All were continuously ponded. Dramatic decreases in rates of infiltration resulted in the control set and the set which received the mulch. Only the mercuric chloride columns maintained infiltration rates near the initial rate, indicating that clogging is largely due to biological activity. In another study, permeability increased when organic matter was first added to the soil, then wetted, incubated, and dried before water was applied (McCalla, 1945, 1946). This increase was attributed to increased soil aggregation. Therefore, McCalla (1950) concluded that under conditions of prolonged submergence, there appear to be two ways by which microorganisms may reduce water movement through the soil. First, the microorganisms may produce gases or organic materials, such as slimes, that may interfere with water movement. Second, microorganisms may reduce water percolation by decomposing or changing agents responsible for stabilizing soil structure, resulting in pedal deterioration and loss of planar voids.

Other investigators also precluded the possibility that microbial cells alone cause clogging. Bendixen, et al. (1950) observed that under continuous inundation, clogging occurred only in the top few centimeters of the soil and not throughout the column. Winneberger, et al. (1960) found that well-aerated water applied at the infiltrative surface of the soil did not prevent the development of an anaerobic soil system under continuous inundation. Since no bacteria were added through the water and no action was involved in concentrating microorganisms at the surface, the clogged layer which developed must have resulted from anaerobic activity on organic matter originally in the soil.

Jones and Taylor (1965) studied the effects of intermittent dosing versus continuous ponding of septic tank effluent on infiltration rates through sand in laboratory columns. Loss of infiltrative capacity occurred relatively soon in the columns continuously ponded while the columns in which aerobic conditions were maintained clogged much slower, occurring in three phases (See Figure B-46). The first phase was attributed to physical blockage of the interstices of the sand by the accumulation of organic deposits because the rate of clogging was directly proportional to the volume of effluent percolated. In the second phase, clogging proceeded at a relatively slow rate, as evidenced by small changes in conductivity over a period of several weeks. It was speculated that a quasi-equilibrium condition was attained where organic losses by decomposition were roughly equivalent to those added by the liquid waste. In the third stage, clogging proceeded relatively rapidly, and the rate of infiltration stabilized near 0.5 to 1.0 percent of its original value. Jones and Taylor concluded that the duration of the first and second phases were dependent upon the application rate and the initial conductivity of the soil. Once into the third stage, clogging rapid independent of the liquid loading or initial conductivity.

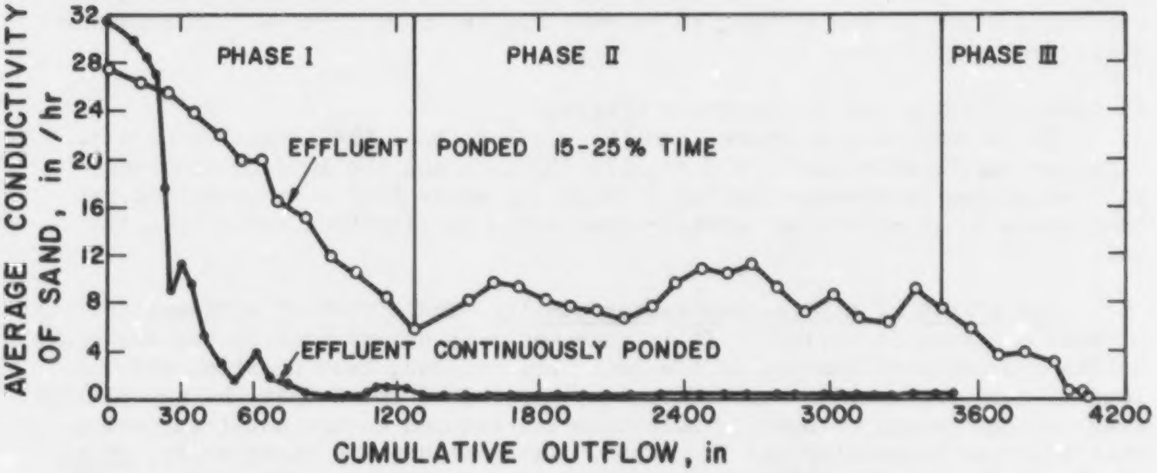


Figure B-46. Typical curve showing the effects of both physical and biological clogging (after Jones and Taylor, 1965).

Jones and Taylor (1965) also found that under continuous ponding, the second phase of clogging was either absent or of short duration. Initially, the clogging rate was directly related to the volume of effluent absorbed.

Later, the rate approached a minimum value which depended on the initial conductivity of the sand. It was concluded that the conditions of Phase II must be maintained by a proper loading rate and pattern, since excessive clogging is inevitable once conditions of continuous ponding are reached.

Thomas, et al. (1966) also observed the three phases of clogging in sand lysimeters dosed with septic tank effluent. Once the third phase was reached, however, waste application was interrupted for a period of time. When loading was resumed, it was found that much of the infiltrative capacity was regained. Upon analyses of the soil, the organic matter was the only probable clogging agent found to have declined.

Since different organic compounds differ significantly in their biodegradability, partially degraded organic material could accumulate with time to clog the soil. While this may contribute to long-term clogging, it does not seem to be the dominating mechanism. Avnimelech and Nevo (1964) and Mitchell and Nevo (1964) investigated the effects of prolonged percolation of water containing high levels of organic matter through sand columns. Initial results showed that polysaccharide-producing microorganisms predominated in the clogged layers of the sand. A direct correlation between accumulation of polysaccharides and polyuronides in the soil and reduction of the infiltrative capacity was found. If application of waste were interrupted long enough for strictly aerobic conditions to return, the polysaccharides and polyuronides were broken down, and much of the infiltrative capacity was restored. On the other hand, if strictly anaerobic conditions were maintained in the column, no polysaccharides were produced and clogging progressed more slowly.

Factors Effecting the Intensity of Clogging--

The accumulated evidence seems to indicate that the intensity of soil clogging may be effected by the organic strength and the loading rate and pattern of the wastewater applied. These are areas that more attention has been given in an effort to maintain reasonable infiltration rates into the soil.

The effect of applied wastewater quality--The effect of effluent quality on soil clogging is unclear. It is reasonable to assume that if the suspended solids and nutrient loading of the soil were reduced, less physical and biological clogging would occur. Weibel et al. (1954) found that the percolation rate through packed columns of silt loam was reduced as the total suspended solids of the wastewater was increased. Later studies by Winneberger, et al. (1960) compared rates of infiltration reduction produced by septic tank and extended aeration unit effluents. The aerobically treated waste had a higher concentration of suspended solids and biochemical oxygen demand than the septic tank effluent, and was found to produce an earlier but less intense clogging in sand. The opposite was true in sandy loam, however. Laak (1970, 1973) conducted similar experiments and found that the equilibrium infiltration rate did not differ between soil types but varied with solids and nutrient loading.

The evidence seemed to indicate that effluent quality may affect the rate or intensity of clogging under certain circumstances but it was still unknown which element in wastewater causes the clogging. Also it was unclear if all soils are equally affected. To further elucidate, several laboratory studies with packed and undisturbed soil columns were conducted.

The first phase of study attempted to test the hypothesis that aerobically treated wastewater causes less intense clogging than septic tank effluent (Daniel and Bouma, 1974). Since reduced infiltration is a more severe problem in slowly permeable soils, a fine textured soil was used. All previous studies used artificially aggregated soil fill materials rather than undisturbed cores in which flow patterns of liquid are significantly different (Bouma and Anderson, 1973). Therefore, to be meaningful, undisturbed cores of soil were extracted in the field.

The soil used in the experiments were Almena silt loam (Aeric Glossaqualf) which has an A2 horizon at 20-33 cm depth with platy structure, and a silt loam texture. The B21t horizon at 33-68 cm depth has medium prismatic structure and the B22t at 68-110 cm has coarse prismatic structure. Both are silty clay loam in texture. Almena is a somewhat poorly drained and slowly permeable soil.

Undisturbed vertical soil cores were taken in the field using a truck-mounted hydraulic probe. The upper end of each core was at about 30 cm depth corresponding with the middle portion of the B21t horizon. Upon sampling, the soil cores were immediately coated with a layer of paraffin-petroleum jelly mixture to prevent structural damage and loss of moisture. Each was placed into lengths of 10 cm diameter plastic pipe, which were later sealed with the paraffin-petroleum jelly mixture. The final dimensions of the soil cores were 10 cm in diameter and 55 cm deep. Eight cm of medium gravel was placed on the upper surface of the soil after the surface had been cleared of loose soil.

The columns were wall-mounted in the laboratory. Tensiometers and platinum electrodes were inserted in the columns at different levels to monitor moisture and redox potentials (See Figure B-47). A constant head of 5 cm of applied wastewater was maintained above the soil surface by mariotte siphons. Suspended solids were kept in suspension by magnetic stirrers.

Duplicate columns were subjected to constant ponding with (1) septic tank effluent, (2) extended aeration effluent, and (3) distilled water amended with sodium, magnesium and sulfate salts to simulate the salt content of septic tank effluent. The chemical oxygen demand and suspended solids concentrations of the extended aeration and septic tank effluents were 60 mg/L COD, 33 mg/L SS and 150 mg/L COD, 40 mg/L SS, respectively.

A gradual reduction in flow occurred in all the columns (See Figure B-48), but comparison of data between columns is difficult due to differences in initial flow rates. There was little difference between the other two sets of columns.

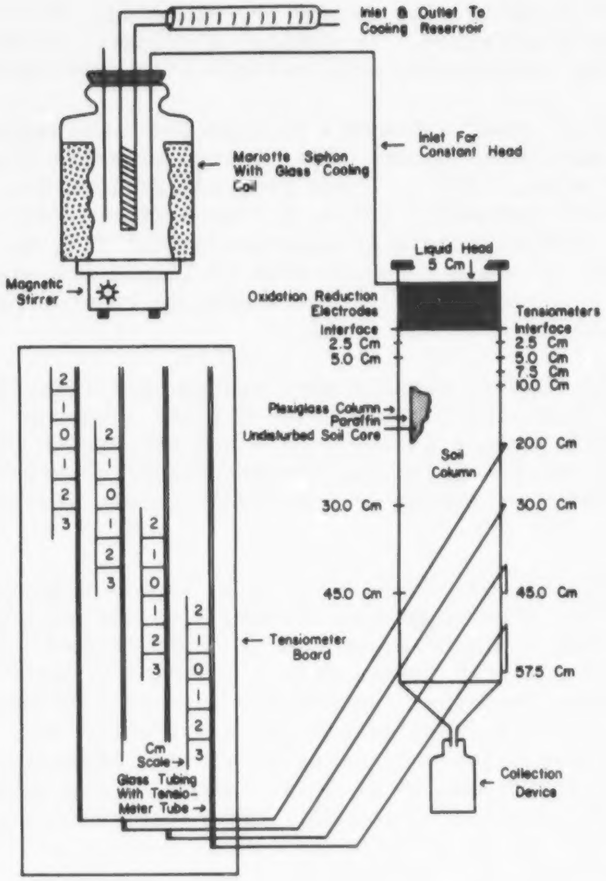

Inlet & Outlet To Cooling Reservoir
Inlet For Constant Head
Mariotte Siphon With Glass Cooling Coil
Magnetic Stirrer
Liquid Head 5 Cm
Oxidation Reduction Electrodes
Interface
2.5 Cm
5.0 Cm
Tensiometers
Interface
2.5 Cm
5.0 Cm
7.5 Cm
10.0 Cm
Plexiglass Column
Paraffin
Undisturbed Soil Core
Soil Column
20.0 Cm
30.0 Cm
30.0 Cm
45.0 Cm
45.0 Cm
57.5 Cm
Tensiometer Board
Cm Scale
Glass Tubing With Tensio-Meter Tube
Collection Device
2
1
0
1
2
3

Figure B-47. Soil column with details of oxidation-reduction electrodes, tensiometers, and cooling system for constant head device (not to scale)

A more meaningful way for comparing relative degrees of clogging is to consider the tensiometric data (Daniel and Bouma, 1974). This provides a measure of the energy status of the liquid in the soil independently of the permeability which varies between columns. The water potential above each column was +5 cm. The rate of decrease of this potential with depth provided by tensiometry gives an absolute measure for the rate of development of surface impedance.

The soil tensions after four months of ponding in the six columns are presented in Figure B-49. The potentials in the columns ponded with distilled water follow very closely the theoretical curves describing uninhibited flow of water under saturated conditions (Figure B-49). The tensions in columns ponded with extended aeration effluent resemble those

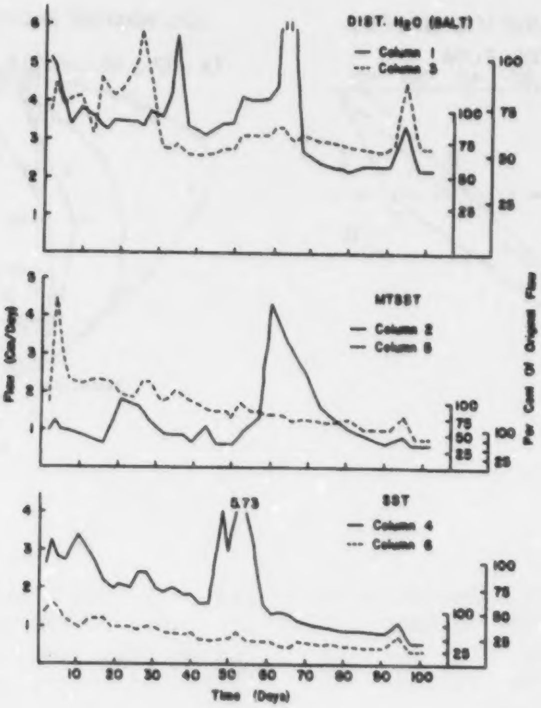


Figure B-48. Reduction in flow rate, in the columns, expressed in cm/day and percent of original.

in the theoretical curve for crusted soil below 10-cm depth. The cores ponded with septic tank effluent have an intermediate position indicating the occurrence of some resistance to flow at the surface, but less than that in columns ponded with the aerobically treated waste.

The difference in physical behavior between the columns loaded with septic tank effluent and those loaded with extended aeration unit effluent may be due to different sizes and shapes of the suspended solids. Freeze dried solids from both effluents were compared and were found to be coarser in the septic tank waste. Finely divided particles in the aerobically treated effluent may have more easily penetrated the relatively porous topsoil to form "bottlenecks" in the pore system with depth, thus reducing the overall permeability (Daniel and Bouma, 1974).

To test further what element in wastewater results in clogging, another series of columns was set up. Undisturbed cores of Almena silt loam were again used. In all, twenty-one columns were constructed and divided into three groups as follows: Group A: seven columns for septic tank effluent; Group B: six columns for extended aeration effluent; and Group C: eight columns for synthetic effluents. Duplicate columns in groups A and B were

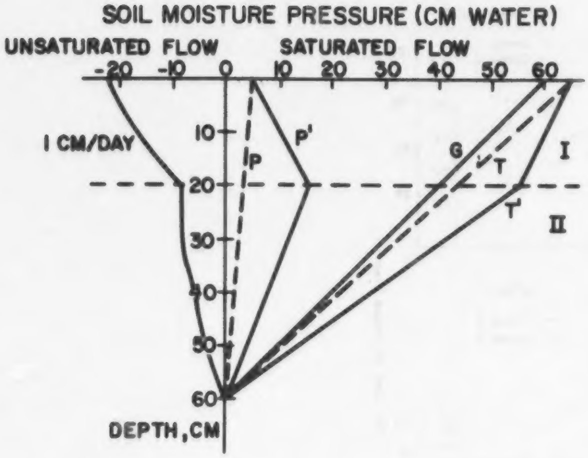


Figure B-49a. Theoretical moisture pressure distributions in the columns assuming saturated flow in a homogenous porous medium (curves P and T) and in a two-layer medium (I=topsoil, II=subsoil) (curves P' and T'). Unsaturated flow (left curve) was calculated for a flow rate of 1 cm/day. P= moisture pressure (cm), T=total potential (cm). G=gravitational potential (cm) (Daniel and Bouma, 1974).

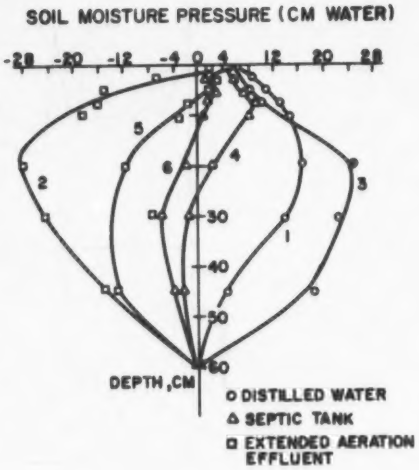


Figure B-49b. Measured moisture pressures in six columns of Almena silt loam, ponded with distilled water, septic tank effluent and aerated effluent (Daniel and Bouma, 1974).

dosed with 1 cm/day (0.2 gpd/ft^2) of raw effluent, sand filtered effluent and paper filtered effluent. Compositions of these liquids are presented in Table B-6. The sand filtered effluents were obtained from two laboratory sand columns (10 x 60 cm) loaded with 5 to 7 cm (1.2 to 1.6 gpd/ft^2) of the respective effluents each day. The paper filtered effluents were prepared by vacuum filtration through Whatman No. 2 filter paper.

Duplicate columns in Group C were subjected to the same dosing regime, but with (1) unsoftened tap water, (2) tap water with glucose and glutamic acid added to obtain a BOD$_5$ at 250 mg/L; (3) tap water with 100 mg/L-N ammonium and 50 mg/L-P phosphate, and (4) tap water with BOD, nitrogen and phosphorus added. There were no solids added in any of the prepared effluents for this group.

The columns were loaded daily for over 200 days. Prior to ponding the rate of clogging appeared to be a function of the initial saturated permeability of the cores which varied between 1 and 6 cm/day. There were no specific trends as a function of effluent quality. When all columns were ponded or about to pond, the columns were continuously loaded.

TABLE B-6. CHARACTERISTICS OF WASTEWATER EFFLUENTS APPLIED TO SOIL COLUMNS

Type of Effluent	BOD_5 mg/L	TOC mg/L	TSS mg/L	VSS mg/L	NH_4^+ mg/L-N	NO_3^- mg/L-N	Total N mg/L-N
Septic Tank	300	75	15	15	50	Tr	55
Sand filtered septic tank	100	20	5	1	1	54	55
Paper filtered septic tank	100	70	10	10	50	Tr	55
Extended aeration	30	20	40	30	1	75	76
Sand filtered extended aeration	20	5	2	1	1	75	76
Paper filtered extended aeration	20	20	1	Tr	1	75	76

Some differences in clogging intensity were observed, but were not striking. The extended aeration effluent clogged more than the septic tank effluent, while sand filtration of both the effluents reduced clogging, but more so with the septic tank effluent. The paper filtered septic tank effluent clogged more than all the septic tank derived effluents. Those columns receiving synthetic wastes did not clog, but the infiltration rates for the columns loaded with tap water with BOD_5 and nutrients added were reduced.

A third series of columns were set up to further investigate the effects of effluent quality, as well as dosing and initial saturated conductivity of the soil on the rate of clogging (Baker, 1976c). Twenty eight columns of undisturbed Almena silt loam were set up at the pilot plant study laboratory (Laboratory Site N). Aerobically treated effluent from the extended aeration unit and septic tank effluent were used for the study. The two wastewaters differed significantly in total suspended solids and oxygen demand. Data showing their mean compositions are presented in Table B-7. Tap water was also used for comparison. The three liquids were then automatically applied to the surfaces of the soil columns at a loading rate of 1 cm/day (0.24 gpd/ft^2) in a single daily dose.

The saturated hydraulic conductivity (K_{sat}) of each column was measured at the commencement of the experiment and after 40 weeks of operation.

TABLE B-7. CHARACTERISTICS OF EFFLUENTS USED FOR THIRD SERIES OF CLOGGING EXPERIMENTS (Baker, 1976c)

	Septic Tank			Extended Aeartion		
	Mean	Standard deviation	Range	Mean	Standard deviation	Range
BOD_5 (mg/L)	48	13	25-91	27	28	2-170
COD (mg/L)	127	26	60-215	80	85	15-690
Total suspended solids (mg/L)	26.5	12	6-74	61	105	11-840
Total nitrogen (mg/L-N)	50.8	14	11-87	44	21	2-84
NH_3 (mg/L-N)	40	18	0-83	0.5	2.4	0-19
Total phosphorus (mg/L-P)	34	9	13-55	34	13	13-96

Measurements were accomplished by ponding tap water on the upper surface for 3 days to saturate the columns and by taking outflow recordings versus time. In the cases where a barrier to flow developed, the conductivity of the clogged column was determined. After this, the upper 1 cm of soil and barrier was carefully removed, and K_{sat} was measured again.

The soil columns were arranged into groups according to their relative initial saturated conductivity ($K_{sat\ I}$). Eight columns were assigned to each effluent, four having similar low $K_{sat\ I}$ values and four having high $K_{sat\ I}$ values. In this way each effluent was applied against a high or low initial conductivity. The four extra columns were assigned to the water subset bringing that group to 13 in all. The major purpose of this design was then to determine whether $K_{sat\ I}$ and effluent quality are major factors in the development of a clogging zone and to see if these parameters influence the length of time required for ponding to occur.

The decision to dose the columns at 1 cm/day was based on the previous column studies and field work. The first series of columns indicated that the long-term infiltration rate was 1.0 cm/day after 4 months of ponding (Daniel and Bouma, 1974). This takes into consideration the loss of area to be expected with gravel "shadowing" a portion of the soil's infiltrative surface.

The reduction in the soil's ability to accept effluent was soon observed in some of the columns, notably in those with low $K_{sat\ I}$ values. Within a month, five columns had liquid continuously ponded on the surfaces. Some of the columns with low $K_{sat\ I}$ resisted ponding for considerable time. Table B-8 shows data for the aerated effluent and septic tank effluent groups. When a dose was initially applied to the soil surface, it ponded on

TABLE B-8. HYDRAULIC CONDUCTIVITY DATA AND PONDING TIMES FOR WASTEWATER-DOSED COLUMNS (Baker, 1976c)

Treatments	Column No.	$K_{sat\ I}$[1]	K_P[2]	$K_{sat\ C}$[3]	Ponding Time
		(cm/day)			(weeks)
Extended Aeration	1	0.51	0.3	0.28	3
	2	0.54	-	0.40	3
	3	0.60	-	0.48	12
	4	0.71	-	0.11	38
	6	1.33	<1	0.54	28
	8	1.89	0.40	0.86	32
	5	1.20	<1	2.14	16
	7	1.43	1.15	1.71	38
Septic Tank	1	.66	0.80	2.85	28
	2	.57	0.96	0.89	24
	3	.52	0.80	0.91	6
	4	.40	0.32	0.75	4
	5	2.00	<1	2.41	34
	8	2.77	-	5.96	not ponded
	6	2.97	<1	1.29	24
	7	2.79	<1	0.21	24

[1] Initial saturated K
[2] K after ponding occurs
[3] Saturated K after clearing surfaces of columns

the surface only temporarily before flowing into the soil. With continued daily applications the amount of time required for the effluent to infiltrate gradually increased until the liquid did not flow away in 24 hours. From this point, on liquid was continuously ponded on the surface, a condition represented here by PT, the ponding time.

Table B-8 shows that all effluent-dosed columns except one, ponded during the course of the experiment. The mean ponding time for the septic group, eliminating the unponded column, was 20.6 weeks (s = 11.2 weeks) and for the aerobic group was 21.3 weeks (s = 14.7 weeks). Hence, the ponding time was nearly the same for the two treatments. However, it should be noted that the mean $K_{sat\ I}$ of the septic group was 1.42 cm/d (s = 1.14 cm/d) and for the aerated columns is 1.04 cm/d (s = 0.51 cm/d). Had they been equal, the difference in PTs may have been greater. In either case only one of 16 columns remained unponded after 10 months of dosing. The K_P values are generally lower than $K_{sat\ I}$ values except for a few receiving septic tank effluent which showed slight gains. After measurements were made, the infiltrative surfaces were removed to a depth of about 1 cm leaving fresh surfaces. These $K_{sat\ C}$ values are roughly the same as at the start of the experiment with the exception of several of the columns which received aeration effluent. It is believed that clearing the surface removed any shallow clogging mat, and also accumulated solids and slaking. Since the extended aeration group did not recover to the extent the septic tank group did, this may indicate that a more in-depth clogging occurred in the extended aeration group, as has been suggested (McGauhey and Krone, 1967; Daniel and Bouma, 1974).

Table B-9 contains information for the water-dosed columns. Several of these became ponded with a mean PT of 18.3 weeks (s = 14.5 weeks) roughly equivalent to those of the effluent-dosed columns. The mean $K_{sat\ I}$ for these ponded columns was 0.90 cm/day, (s = 0.12 cm/day). In this case, the lower mean $K_{sat\ I}$ probably accounts for lower mean PT. As in the other columns, K_P generally decreased, except for column 6. After clearing of the surface, conductivities returned to initial or to slighly higher values, an indication that the restrictive layer was at the surface, probably due to slaking. The results of the ponding time data are summarized in Table B-10. The difference in the length of time to ponding for the different applied liquids is not significant. There appears, however, to be a relationship between $K_{sat\ I}$ and PT. Table B-10 examines this relationship by grouping all ponded columns into high and low groups of initial K_{sat}. Although the individual data are highly scattered, a significant difference in ponding time for the two groups is clear. This indicates that for a 1 cm/day dose of liquid, higher $K_{sat\ I}$ will in general lead to a longer acceptance time, but for the Almena silt loam used here, this gain is only a few months.

Five of the columns dosed with water and one dosed with septic tank effluent did not pond in the 40 week period. Their conductivities actually increased with dosing before and after surface clearing. No visible surface crust had developed, except for very slight slaking of a few surfaces. Interpedal cracks and root channels were open to the surface allowing the free flow of water into the column. Rapid movement of water through these pores appears to have enlarged the flow paths, increasing their conductance. Morphological evidence of this change was visible when the columns were examined in detail. This may be viewed as a form of adaptation of the soil morphology to the new moisture regime of pulse-loading 365 cm/year, versus the 70-80 cm/year found naturally. A decrease in the daily minimum potential (Ψ_m min) with continued dosing indicates that flow has increased through larger channels without having time to disperse into the pedal interiors (Figure B-50). The ped interiors have in effect become more well-drained.

TABLE B-9. DATA FOR WATER-DOSED COLUMNS, (Baker, 1976c)

Treatments	Column No.	$K^{1}_{\text{sat I}}$	K^{2}_{P}	$K^{3}_{\text{sat C}}$	Ponding Time
			(cm/day)		Weeks
Ponded columns	1	0.74	0.60	1.07	10
	2	0.89	0.9	2.25	32
	3	0.97	0.7	0.70	12
	4	1.04	0.95	1.04	36
	6	0.96	14.7	15.7	32
	12	0.96	1.5	1.61	3
	13	0.71	0.32	0.79	3
			K_{sat} (before clearing surface)		
Unponded	5	1.02	3.59	3.65	not ponded
	7	0.76	6.43	7.40	"
	8	0.72	5.14	5.79	"
	10	0.29	3.0	56.6	"
	11	0.06	7.82	5.79	"

[1] Initial saturated K

[2] K after ponding occurs

[3] Saturated K after clearing surfaces of columns

Analysis of soil moisture potentials recorded by tensiometers at depths below the surface sheds light on happenings within the columns. Figure B-51 represents the theoretical soil moisture profile for three possible situations in a two-layered soil, such as Almena silt loam with a more permeable upper horizon. Curve (a) represents saturated flow through the column, where potentials are high in horizon I. Curve (b) represents a steady-state flow through a restrictive barrier just below the soil surface (the upper portion of the column is uniformly drained). Curve (c) represents the calculated potentials expected at a constant flow rate of 1 cm/day (Bybordi, 1968) using the K-curves for these two horizons. In natural soils the change in conductivity with depth may not be as well defined as in this example but it is often characterized by gradually decreasing with depth to the least permeable horizon.

TABLE B-10. SUMMARY OF PONDING TIME DATA AS RELATED TO TREATMENT AND INITIAL HYDRAULIC CONDUCTIVITY, (Baker, 1976c)

a)

Treatment	Mean $K_{sat\ I}$ (cm/d)	Ponding Time[1] (weeks)	
Extended Aeration	1.03, (s = 0.51)	$\overline{PT}$ = 21.3	s = 14.7
Septic Tank	1.42, (s = 1.14)	$\overline{PT}$ = 20.6	s = 11.2
Water	0.90, (s = 0.12)	$\overline{PT}$ = 18.3	s = 14.5
Extended Aeration and Septic Tank		$\overline{PT}$ = 20.9	s = 12.7

b)

Conductivity Class	Ponding Time (weeks)	
Low $K_{sat\ I}$ (0.56 cm/d mean)	$\overline{PT}$ = 14.8	s = 13.5
High $K_{sat\ I}$ (1.94 cm/d mean)	$\overline{PT}$ = 28.0	s = 7.4

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[1] $\overline{PT}$ = mean ponding time, S = standard deviation

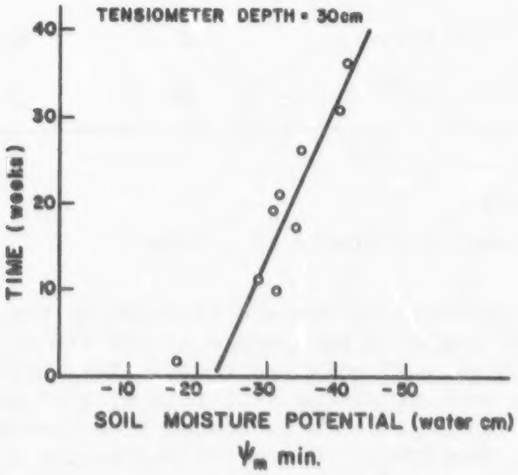

Figure B-50. Plot of PT (ponding time) against Ψ_m min, (daily minimum moisture potential) for water column 8 (Baker, 1976c)

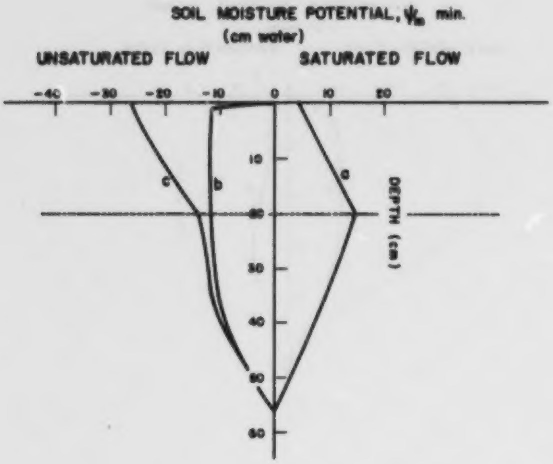

Figure B-51. Calculated soil moisture potential profiles for three column situations. Curve (a) represents an overloaded column, (b) a column in which a clogging zone has developed restricting flow, and (c) a non-ponded column receiving 1 cm/day of liquid (Baker, 1976c)

Figure B-52 presents the soil moisture profile for three different columns chosen to represent the three possible outcomes outlined above after 38 weeks of dosing at 1 cm/day on Almena silt loam columns. The values of Ψ_m (matric potential) used are the driest or minimum potentials reached that day during the dosing cycle. Therefore, they represent the potential after 24 hours of drainage following the last dose. When these Ψ_m values are plotted against subsurface depth, the resulting curves are similar to those of Figure B-51. Each curve clearly illustrates the properties operating within the columns. Column a' (Septic 2) had liquid ponded on its surface because the loading rate of one cm/day exceeded the column's $K_{sat\ I}$. Some clogging may have occurred, but the dominant effect expressed here is excessive loading. It took 24 weeks for continuous ponding to occur. Column b' (Extended aeration 5) clearly demonstrates the formation of a barrier to flow near the surface, with ponding occurring at 16 weeks. In this case, a uniform negative potential indicates the maintenance of unsaturated conditions. Column c' (Water 8) did not pond, and the potentials are even more negative than those predicted by calculation. The soil is more well-drained which can be explained very simply by flow through a variety of pore sizes. In the case of a daily dose, liquid is rapidly conducted away by the larger pores such as interpedal cracks and root channels before much water enters the finer pores or soil peds. The tensiometers record the potential of the ped interiors and of the many small pores not exposed to the dose. In the non-ponded columns the potential increased over the course of the study, as did their K_{sat} values. This is easily explained by the enlargement of the major flow pathways, allowing even faster conduction of liquid and reducing further the potentials within peds.

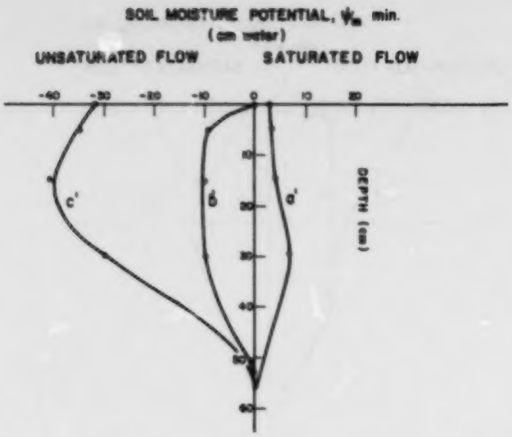

Figure B-52. Observed soil moisture potential profiles for 3 columns. Curve (a') is septic column 2; (b') is aerated column 5; and (c') is non-ponded water column 8. Measurements were made after 38 weeks of dosing at 1 cm/day. (Baker, 1976c)

The evidence from the three series of column studies indicates that some substance in wastewater which is not present in tap water causes soil clogging. This substance has not been identified. However, it is obvious that aerobic treatment does not remove it entirely. Partial removal must occur, for intermittent sand filter runs are significantly longer when aerobically treated wastewater is applied in comparison to runs of filters receiving septic tank effluent (See Appendix A). This same effect is not seen in finer textured soils, as demonstrated by the laboratory studies. Flow through these soils seems to be controlled more by the original hydraulic conductivity of the soil than by the resistance of the clogging mat.

The effect of dosing and resting--Following the work of Bendixen, et al. (1950), Winneberger, et al. (1960), Jones and Taylor (1965) and Thomas, et al. (1966) it has been generally thought that biological clogging could not predominate, provided the soil absorption system remained at least intermittently aerobic. Early laboratory work with lysimeters showed repeatedly that reduction in the infiltrative capacity of the soil proceeded more slowly when periods of ponding were interrupted with periods of aeration. These findings have motivated several working concepts in septic tank practice that are meant to confer greater longevity on the system, such as installation of vent pipes to facilitate reaeration of the soil after a wetting front has passed, use of aerobic treatment units, use of dosing systems for controlled effluent application, and periodic resting of absorption systems by the use of twin disposal systems. Some of these concepts are so new in practice that there has not been enough time to establish their validity under field conditions.

Contrary to findings of earlier workers, Kropf, et al. (1975) reported that infiltration rates through constantly ponded soil columns remained higher than those in columns subjected to intermittent flooding. In an earlier study, Mitchell and Nevo (1964) correlated clogging with polysaccharide accumulation in soil. The polysaccharides were produced by a facultative bacterium of the genus *Flavobacterium* which would be favored under short periods of alternating aerobic and anaerobic conditions produced by intermittent dosing. Under strictly anaerobic conditions, no polysaccharides were produced. If polysaccharides are a dominant clogging agent, this might explain the results of Kropf, et al.

The results of these two investigations imply that the oxidation-reduction potential in and around the clogging mat may be critical to maintaining high infiltration rates. Totally aerobic conditions seem to promote rapid degradation of the clogging materials, but totally anaerobic conditions seem to prevent the buildup of an excessively resistant mat. Under fluctuating anaerobic and aerobic conditions created by intermittent dosing, polysaccharide-producing facultative organisms could be favored. Therefore, if dosing and resting is to be used as a means for prolonging the life of soil absorption fields, more must be known about the growth and degradation dynamics of the clogging mat.

Much of the information on soil clogging has been obtained from column studies designed to simulate a natural field clogging regime. Generally, the studies use lysimeters with air tight walls, which create anaerobic conditions within the soil below the clogging mat. Walker, et al. (1973a) has shown that aerobic conditions exist under soil absorption systems located in sand. Thus, most of the previous research has neglected to acknowledge the impact of redox conditions in designing laboratory experiments to evaluate the dynamics of soil clogging.

Recent work has shown that clogging mechanisms in aerated and non-aerated columns can be quite different. Magdoff, et al. (1974a, 1974b) showed that clogging was not delayed in aerated sand columns. The resistance of the clogging mat in soil columns which were aerated by perforating the column sidewall were shown to more closely reproduce the resistances measured below four clogged absorption fields in sands (Magdoff and Bouma, 1974). Thus, some of the reported work studying the dynamics of soil clogging is of limited applicability to determining acceptable loading patterns.

To obtain the necessary information, column experiments were designed which included column aeration as a variable (Perry and Harris, 1975). The objectives of this study were to evaluate the dynamics of soil clogging caused by continuous ponding with septic tank effluent and the dynamics of infiltration rate recovery upon resting. Respirometric techniques were also used to determine decomposition kinetics of the clogging mat to aid in identifying parameters affecting the restoration of the infiltrative surface.

Eighteen soil columns were constructed by uniformly packing plexiglass columns (10 cm diameter x 60 cm long) with 50 cm of sand (U.C. = 1.99, E.S. = 0.14) obtained from the C horizon of a Plainfield loamy sand (Typic Udipsamment) (See Figure B-53). Saturated hydraulic conductivity (K_{sat}) values were approximately 500 cm/day which represented field conditions. Stones were placed on the sand surface to prevent disruption of the surface crust during effluent application. Soil water tension was monitored with a flow-through tensiometer placed 5 cm below the soil surface.

To simulate the natural aerobic field situation beneath a seepage bed, 9 columns were subcrust aerated (SA) by perforating them with numerous 3 mm diameter holes distributed 5 to 10 cm below the sand surface. The remaining 9 columns were maintained under anaerobic conditions and designated nonsubcrust aerated (NSA).

Fresh septic tank effluent was obtained weekly from a residential household and stored in the laboratory at 4°C until used. Average effluent characteristics over the length of the study are presented in Table B-11. To apply the waste liquid, a Mariotte siphon apparatus was used to maintain a constant head of 3 cm over the column surface.

During the clogging phase, effluent was applied at a rate of 127 cm/week (4.5 gpd/ft^2), until such time that clogging had intensified, reducing flow rates. At that time, six of the columns were subcrust aerated. Continuous effluent application was then initiated in all columns. The columns were

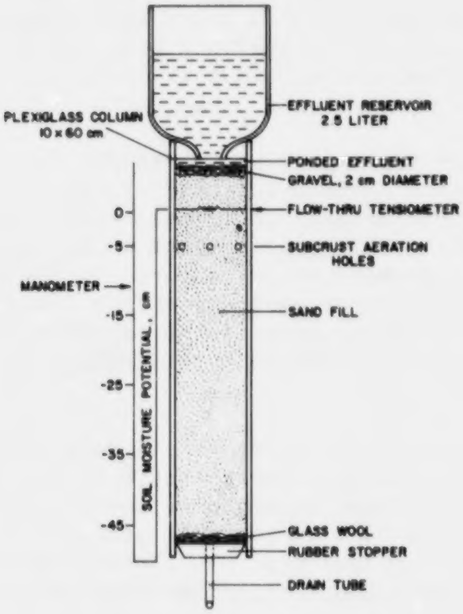
PLEXIGLASS COLUMN
10 x 60 cm
EFFLUENT RESERVOIR
2.5 LITER
PONDED EFFLUENT
GRAVEL, 2 cm DIAMETER
FLOW-THRU TENSIOMETER
SUBCRUST AERATION HOLES
MANOMETER
SAND FILL
SOIL MOISTURE POTENTIAL, cm
0
-5
-15
-25
-35
-45
GLASS WOOL
RUBBER STOPPER
DRAIN TUBE

Figure B-53. Soil column schematic for respirometric studies of soil clogging (Perry and Harris, 1975)

TABLE B-11. SEPTIC TANK EFFLUENT CHARACTERISTICS USED IN RESPIROMETRIC STUDIES (Perry and Harris, 1975)

Property	Mean	Range
	mg/L	
Chemical oxygen demand	257	113 - 394
Biological oxygen demand	134	85 - 220
Total organic carbon	104	25 - 154
Total inorganic carbon	108	81 - 105
Volatile suspended solids	28	7 - 60
Total suspended solids	42	7 - 72
Total volatile solids	303	140 - 460

maintained under this regime for an additional 7 to 9 months. After 8 to 10 months of clogging, infiltration rates were measured using tensiometry and cumulative inflow and outflow from the columns. The rates were found to be 1 to 1.5 cm/day. At this time the unclogging phase was initiated whereby effluent was no longer applied and columns were allowed to aerate. Infiltration recovery was monitored by adding effluent and measuring subcrust water potential and inflow and outflow rates.

Also during this time samples of the clogging mat were removed from the replicate NSA and SA columns analysis. Organic carbon was determined by the autoclave, mercuric sulfate and silver sulfate modification of the reflux dichromate oxidation method (Allison, 1965; Standard Methods, 1971; Unluturk, 1974). Processing of crust samples for subsequent O_2 uptake analysis involved sieving, gravimetric measurement, moisture adjustment to 50% maximum retentive capacity and reaction flask equilibration. Samples were incubated at 22°C in a Gilson Differential Respirometer, Model SGR 20. Nitrogen analysis was performed on the crust samples both before and after incubation to correct oxygen uptake for nitrification: Total N was measured by the #2 simimicro Kjeldahl digestion procedure (Bremner, 1965a), and NH_4-N, NO_3-N and NO_2-N by steam distillation (Bremner, 1965b).

The surface crust was removed in 1-cm sequential units and infiltration through the freshly exposed surface was measured each time. This enabled isolation of the regions within the crust responsible for clogging. The complete history of a single column is shown in Figure B-54.

Typical loading and flow characteristics for NSA and SA columns during the clogging phase are presented in Figure B-55 and Tables B-12 and B-13. The initial response to effluent application was a rapid reduction in infiltration, followed by an increase in flow rate of short duration at 30 days and gradually culminating in an asymptotic decline in infiltration.

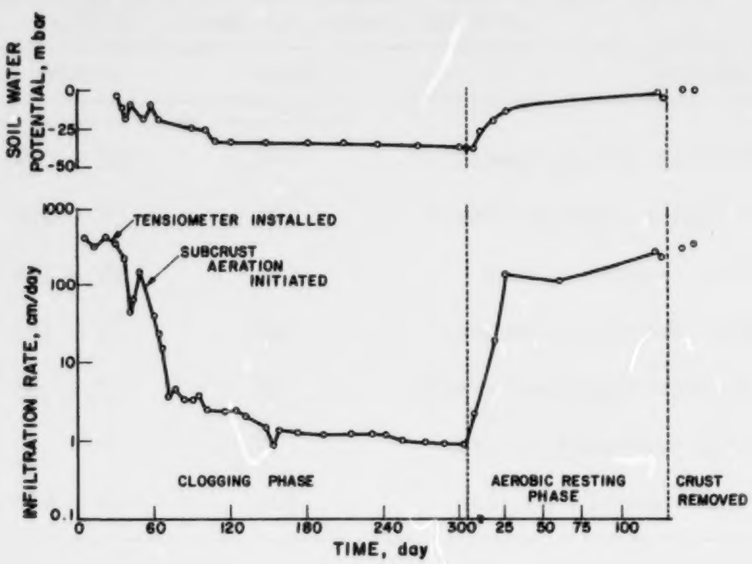

Figure B-54. Soil water potential and infiltration rate during clogging, resting and crust removal phases for a single column (Perry and Harris, 1975)

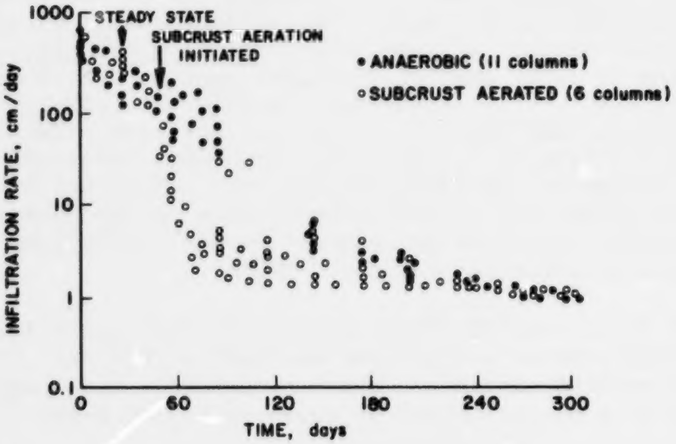

Figure B-55. Infiltration rate reduction during clogging phase (Perry and Harris, 1975)

TABLE B-12. EFFECT OF CONTINUOUS EFFLUENT PONDING ON FLOW CHARACTERISTICS OF A SUBCRUST-AERATED COLUMN OF PLAINFIELD SAND (COLUMN 3) (Perry and Harris, 1975)

	Cumulative load[1]					
Time	Effluent	BOD	VSS	FSS	Flow rate	Subcrust moisture potential
days	cm	mg	mg	mg	cm/day	mbars
1	26				545	--
7[2]	127				415	--
14	254				225	--
21	381				321	--
28[3]	508	4000	520	120	266	--
35	635				120	--
42	762				6.8	36
49	889				73	30
56[4]	1016	8067	880	120	24	42
63[5]					5.4	--
70	1096				2.3	--
77	1110				2.2	--
84	1125				2.1	24
91	1138	8977	1100	130	1.6	29
98	1152				2.1	28
105	1162				1.4	30
112	1171				1.3	31
119	1181	9483	1263	232	1.4	32
126	1191				1.4	31
133	1203				1.8	30
140	1227				3.8	26
147	1259				4.2	27
154	1288	11163	1624	366	2.9	28
161	1313				4.2	24
168	1327				2.1	28
175	1350				3.3	25
182	1364				2.1	--
189	1377				1.8	29
196	1389				2.0	29
203	1398				1.3	31
210	1407				1.4	30
217	1416				1.2	31
224	1422				1.0	32
231	1430				1.2	31
238[6]	1436	12615	1998	512	1.0	33

[1] BOD: Standard 5-day biochemical oxygen demand
VSS: Volatile suspended solids.
FSS: Fixed (inorganic) suspended solids.

[2] Appearance of black vertical streaks below crust, 8-12 cm in length.

[3] Subcrust tensiometer installed.

[4] Subcrust aeration initiated.

[5] Continuous effluent infiltration initiated.

[6] Aerobic rest regime initiated.

TABLE B-13. EFFECT OF CONTINUOUS EFFLUENT PONDING OF THE FLOW CHARACTERISTICS OF A NONAERATED COLUMN OF PLAINFIELD SAND (COLUMN 11). (Perry and Harris, 1975)

	Cumulative load[1]					
Time	Effluent	BOD	VSS	FSS	Flow rate	Subcrust moisture potential
days	cm	mg	mg	mg	cm/day	mbars
1	26				525	
7	51				450	
14	102				320	
21	152				200	
28	203	1514	144	0	100	
35	330				90	
42	457				100	
49	584				50	
56	711	7496	1064	44	35	
63	838				53	34
70	965				36	34
77	1092				18	32
84	1219	15471	3463	1544	17	37
91	1346				18	38
98	1473				27	38
105	1600				16	39
112	1727				15	41
119	1805		5016	2122	5.9	43
126	1932				13	43
133	1963	20100			4.3	43
140	1993				4.2	38
147	2010				2.6	44
154	2027				2.2	44
161	2040				1.7	44
168	2052				1.7	45
175	2066				2.0	46
182	2078				1.8	45
184	2090				1.7	46
196	2103				1.8	46
203	2158	24685	6216	2783	1.9	46

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[1] BOD: Standard 5-day biochemical oxygen demand.

[2] VSS: Volatile suspended solids.

[3] FSS: Fixed (inorganic suspended solids).

The typical S-shaped curve (Figure B-55) resulting when infiltration is plotted against time, has been previously identified by Allison (1947), McGauhey and Winneberger (1965) and Robeck, et al. (1964).

Six columns were subcrust aerated when continuous ponding occurred to see what effect lateral O_2 diffusion had on crust development. Prior to the time of column perforation, a black ferrous sulfide coloration occurred throughout the column. In response to aeration, the black coloration disappeared except for the upper 0.5-cm crust layer. This was followed by the formation of an amorphous light brown layer, 0.5 cm thick with a sharp irregular boundary, directly beneath the upper black surface layer. There was an immediate abrupt decrease in the infiltration rate (Table B-12 and Figure B-55) commensurate with the formation of this brown amorphous layer. For the duration of the study, there was no change in the appearance of the columns. In contrast, the NSA columns remained black and underwent a more gradual decline in infiltration (Table B-13 and Figure B-55). These results indicate that the initial response to aeration is intense clogging, probably resulting from the production of microbial slimes and/or biomass or the formation of oxidized iron compounds directly below the black surface crust layer.

Perforated studies by others (Magdoff, et al., 1974a) indicate that subcrust moisture tensions in columns are lower than in anaerobic nonperforated columns at the time of initial clogging. The more intense crust development within the initial clogging phases of aerated columns is consistent with the observation by Perry and Harris (1975) of lower infiltration rates in aerated columns. These contrasting results between SA and NSA systems also show that an effluent application regime that involves alternating aerobic-anaerobic conditions on a short term basis, causes rapid clogging and that infiltration capacity can be extended by maintaining an anaerobic environment. This is consistent with observation by Kropf, et al. (1975) that continuously flooded soils, more often than not, infiltrated more effluent than intermittently flooded soils.

Figure B-56 shows the cumulative effluent load for SA and NSA columns during the clogging phases. In the 6 month period immediately after subcrust aeration, the NSA columns processed 3 times more effluent volume than the SA columns.

An equilibrium infiltration rate of 1 to 1.5 cm/day was approached in both SA and NSA systems at 5 and 7 months, respectively. These infiltration rates were maintained throughout the duration of the clogging phase, aside from occasional crust breakthroughs. These rates are consistent with values reported by other researchers (Jones and Taylor, 1965; Kropf, et al., 1975; McGauhey and Krone, 1967; and Thomas, et al., 1966) and are independent of aerobic or anaerobic conditions.

The O_2 uptake by samples taken from the clogging zone followed a general pattern that can be divided into 3 periods. There is an initial period consisting of an immediate high rate of O_2 uptake followed by a second period of rapidly declining O_2 uptake. The final period consists of

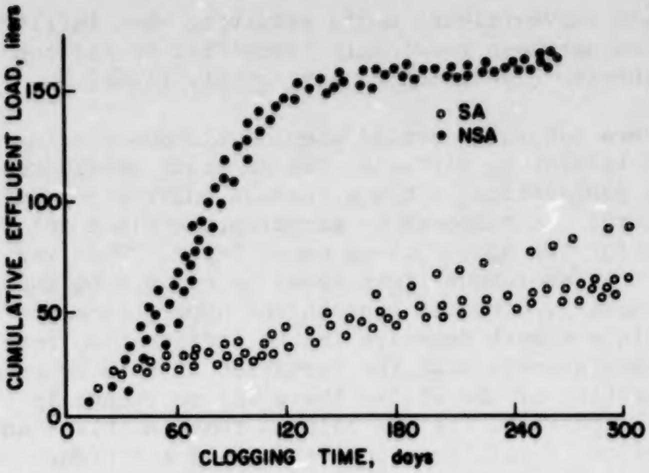

Figure B-56. Cumulative effluent loading for NSA and SA columns (Perry and Harris, 1974)

a very low, but uniform rate of O_2 uptake. This general sequence is consistent with respirometric observations by others evaluating soil metabolic activity (Bhaumik and Clark, 1947; Parr and Norman, 1964) and sewage decomposition (Jenkins, 1960; Ludwig, et al., 1951). In Figure B-59 it can be seen that the initial period generally lasted less than 24 hours, the duration being longer for those samples with higher concentrations of organic carbon. During this period,the intense O_2 uptake is usually attributed to the metabolism of readily available carbon. Crust samples from NSA columns showed a very high rate (80 µl/gm/hr) of O_2 uptake. In contrast, the SA sample had a lower initial uptake rate of less than 20 µl/gm/hr. Large and rapid changes in rates of O_2 uptake were common during this period. Higher maximum rates of uptake were observed from samples containing greater organic carbon. These initial O_2 uptake characteristics indicate that the microbial population is abundant, active, and immediately capable of utilizing the available substrate. Most importantly, data by samples obtained from anaerobic clogging regimes lasting 200 days indicate the facultative biomass was responsible for the immediate, high initial O_2 uptake. Further, this reveals the absence of microbial inhibition which is often associated with the accumulation of toxic reduced sulfur compounds from sewage organics under anaerobic conditions.

The second period lasted 3 to 4 days, during which time O_2 uptake by both NSA and SA samples decreased rapidly. The rate of decline tended to be greatest within samples of higher initial organic carbon content. Toward the later portion of this period, rates approached a lower maintenance level of O_2 uptake. It is generally assumed that the initiation of the second period is due to the exhaustion of an essential or readily available nutrient. The final period consists of a uniformly low rate of O_2 uptake

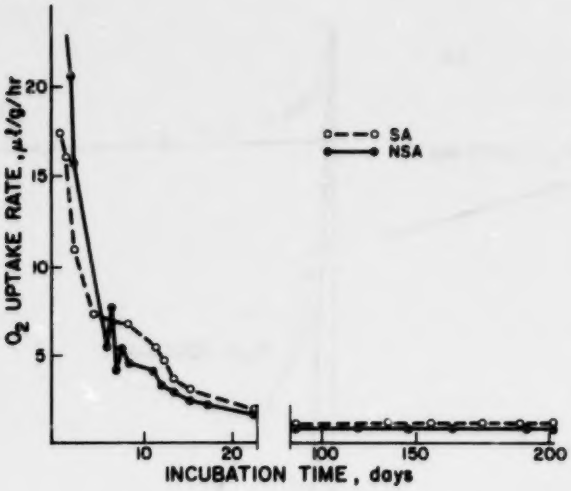

Figure B-57. Rate of O_2 uptake by crust samples from NSA and SA columns (Perry and Harris, 1974)

and may reflect the maintenance level of metabolic activity. The rate curve shows an assymptotic decline during this period and tends to approach the level of endogenous O_2 uptake at approximately 60 days. Millar, et al. (1936) observed that rates by amended systems approached those of endogenous systems between 120-190 days.

First-order organic carbon decomposition kinetic data (McCabe, 1960; Eckenfelder, 1960), extrapolated from O_2 uptake data based on initial organic carbon content (Tables B-14 and B-15) are plotted in Figure B-58. Rate constants for different biodegradable fractions were similar to those obtained from previous incubation studies using crust samples from clogged columns under a variety of conditions (Daniel and Bouma, 1974; Magdoff, et al., 1974a; Magdoff, et al., 1974b; Unluturk, 1974). However, it was not possible to relate different biodegradable fractions to any specific infiltration trend because of saturated hydraulic conductivity variability between columns.

Figure B-59 shows the cumulative O_2 uptake for SA and NSA column crust samples during the resting phase. The decomposition is extrapolated from O_2 uptake data, corrected for nitrification, and based on initial organic carbon and an assumed respiratory quotient of 1. The cumulative values reflect the high initial rates of O_2 uptake.

The infiltration rates during the resting sequence are presented in Figure B-60. All columns showed a 100 fold increase in the infiltration rate within the initial three weeks of resting. These infiltration trends are consistent with values reported in the literature (Thomas, et al., 1966).

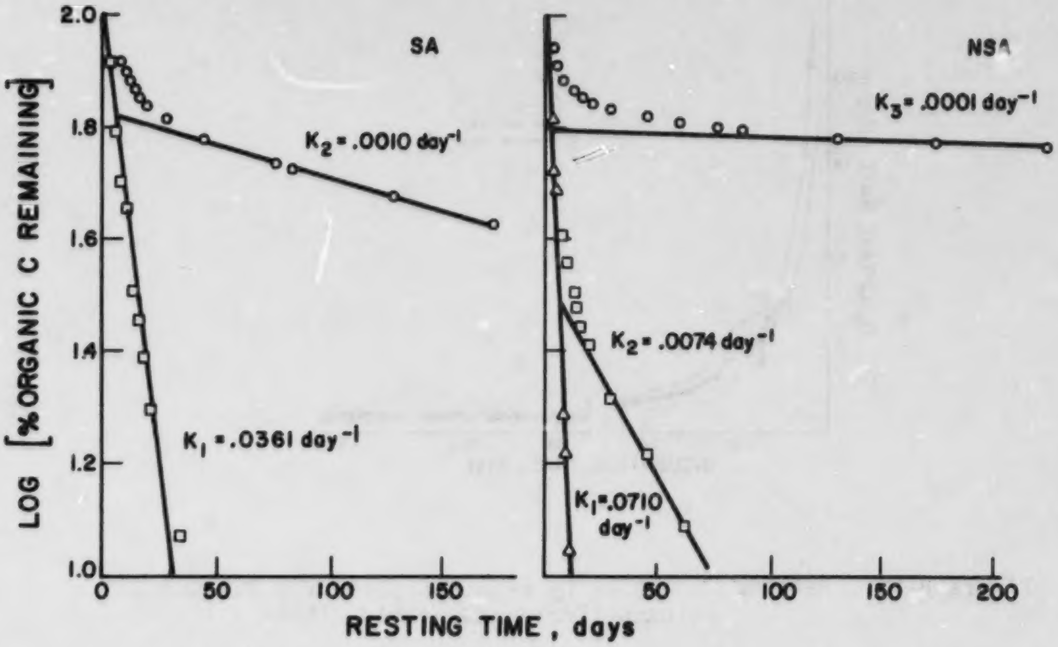

Figure B-58. First order organic carbon decomposition for SA and NSA columns (Perry and Harris, 1975).

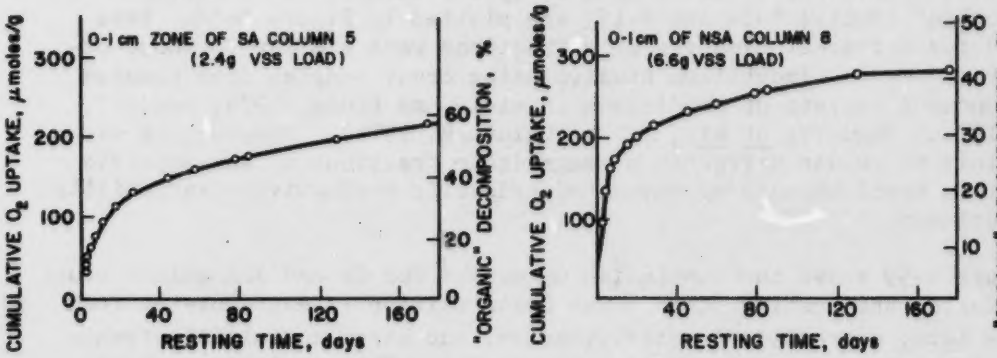

Figure B-59. O_2 uptake rate by samples from SA and NSA columns during resting phase (Perry and Harris, 1975).

TABLE B-14. O_2 UPTAKE BY CRUST SAMPLES FROM A SA COLUMN DURING RESTING (Perry and Harris, 1975)

	Oxygen uptake		Cumulative organic C decomposition		
Incubation time	Rate	Cumulative	Total amount decomposed	Amount left of initial organic C[1]	
hr	10^{-3}μm/g/hr	μm/g	mg/g soil	%	Log %
5	1143	6.0	.07	98.4	1.99
6	911	6.5	.08	98.3	1.99
8	969	8.4	.10	97.8	1.99
11	754	10.4	.12	97.3	1.98
26	696	20.9	.25	94.6	1.97
27	616	21.4	.26	94.5	1.97
49	464	33.4	.40	91.3	1.96
97	303	51.9	.62	86.5	1.93
100	295	52.9	.63	86.2	1.93
144	277	65.3	.78	83.0	1.91
145	304	65.7	.79	82.6	1.91
148	286	66.6	.80	82.6	1.91
169	295	69.8	.83	81.8	1.91
176	281	71.9	.86	81.2	1.90
186	263	74.6	.90	80.5	1.90
194	277	76.6	.92	80.0	1.90
270	219	95.4	1.14	75.3	1.87
271	165	95.7	1.15	75.1	1.87
289	152	98.5	1.18	74.3	1.87
292	183	98.9	1.19	74.2	1.87
306	138	101.3	1.21	73.6	1.86
312	129	102.0	1.22	73.4	1.86
313	125	102.2	1.23	73.4	1.86
315	147	102.5	1.23	73.4	1.86
317	134	102.8	1.23	73.2	1.86
319	156	103.0	1.24	73.2	1.86
322	129	103.4	1.24	73.1	1.86
330	125	104.5	1.25	72.8	1.86
336	116	105.2	1.26	72.6	1.86
361	111	108.0	1.30	71.8	1.85
440	89	115.9	1.39	69.8	1.84
652	63	132.1	1.59	65.6	1.81
1058	38	152.3	1.83	60.4	1.78
1395	25	162.9	1.95	57.6	1.76
1843	24	173.1	2.08	54.9	1.74
1940	19	175.2	2.10	54.3	1.73
2022	22	176.9	2.12	53.9	1.73
3096	20	199.4	2.39	48.2	1.68
4200	17	219.6	2.64	42.8	1.63
5423	15	238.7	2.84	37.8	1.58

[1] Initial organic carbon = 4.60 mg/g.

TABLE B-15. O_2 UPTAKE BY CRUST SAMPLES FROM A NSA COLUMN DURING RESTING (Perry and Harris, 1975)

Incubation time	Oxygen uptake Rate	 Cumulative	Cumulative organic C decomposition Total amount decomposed	 Amount left of initial organic C[1]	
hr	10^{-3} μm/g/hr	μm/g	mg/g soil	%	Log %
6	2419	20.0	0.24	97.1	1.98
26	2205	66.3	0.80	90.0	1.96
49	661	94.1	1.13	86.3	1.94
97	1009	133.2	1.60	80.6	1.91
148	290	163.3	1.96	76.2	1.88
170	156	168.2	2.02	75.4	1.88
192	165	172.3	2.07	74.8	1.87
270	156	184.8	2.22	73.0	1.86
321	103	191.0	2.29	72.1	1.86
331	94	191.9	2.30	71.9	1.86
350	85	193.6	2.32	71.8	1.86
369	80	195.2	2.34	71.5	1.85
429	76	199.8	2.39	70.8	1.85
448	63	201.1	2.41	70.6	1.85
652	54	213.0	2.56	68.9	1.84
1058	38	231.4	2.78	66.2	1.82
1395	28	242.4	2.91	64.6	1.81
1843	24	254.0	3.05	62.9	1.80
1940	18	256.0	3.07	62.6	1.80
2022	23	257.6	3.09	62.4	1.79
3096	15	278.2	3.34	59.4	1.77
4200	11	282.5	3.39	58.8	1.77
5423	8	293.4	3.52	57.2	1.76

[1] Initial organic carbon = 6.85 mg/g.

No increase in infiltration rates occurred during a 1 cm sequential removal of 6 cm of surface crust in the NSA columns. The apparent in-depth crust development is a result of anaerobic conditions and/or the higher amounts of effluent application prior to resting. In contrast, removal of the 0-1 cm surface crust in SA columns resulted in an additional 13 percent increase in the infiltration rate, showing that thin crust with high crust resistance developed within the surface region of aerobic columns.

The results of this study indicate that substantial differences exist regarding the nature and mechanisms involved in clogging and resting-induced infiltration surface restoration between aerated and non-aerated columns. This has implications with respect to extrapolation of data and conclusions derived from non-aerated column experiments to field conditions in coarse textured soils where aeration below the clogging mat usually prevails. Non-aerated columns clogged slower than aerated columns but infiltration

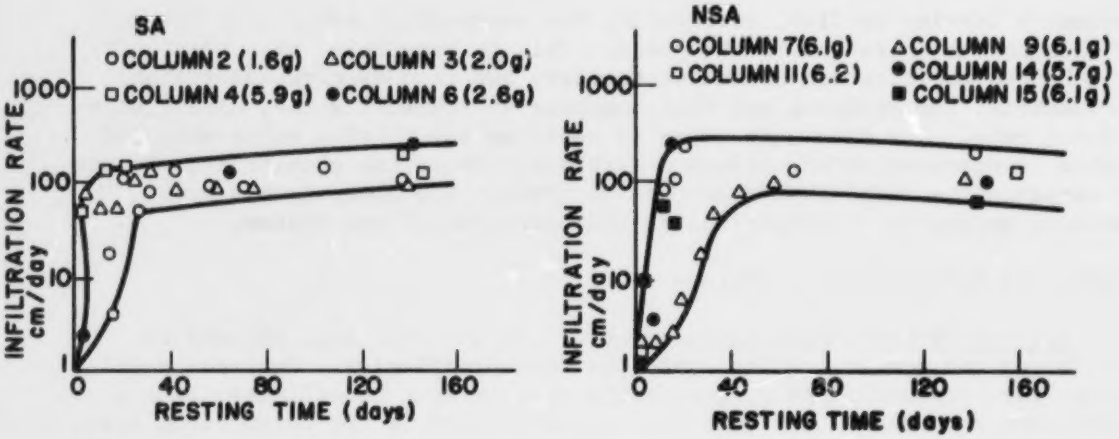

Figure B-60. Infiltration rate recovery for SA and NSA columns during resting phase. (Values in parentheses are cumulative volatile suspended solids load for the respective column) (Perry and Harris, 1975)

rate recovery during resting was more rapid in the aerated columns (Perry and Harris, 1975). Therefore, extrapolation of data from laboratory columns for recommending design and operational requirements of soil absorption fields must be done with care.

The aerated column studies indicate that effluent application regimes, characterized by alternating anaerobic-aerobic conditions on a short term basis (dosing and short cycle alternating seepage systems),could result in reduced infiltration associated with the formation of an intense crust directly below the surface which develops during the aerobic (resting) phase. Once clogged, restoration of the infiltrative surface by resting requires at least 3 to 4 weeks in sands. This appears to be sufficient time to decompose 30 to 35 percent of the organic carbon present in the mat and reestablish acceptable infiltration rates (Perry and Harris, 1975). This implies that dosing and resting frequencies must be selected with care to prevent excessive clogging. It is yet unknown what these frequencies should be.

Design of Soil Absorption Systems

If a septic tank-soil absorption system is to operate satisfactorily, the soil must continue to absorb wastewater at an acceptable rate over a reasonable length of time. This would be a simple matter, if the pores in the soil would remain open, but when wastewater is applied to the soil, a clogging mat often forms at the infiltrative surface. The clogging mat becomes a barrier to flow, restricting the movement of water into the soil by closing the entrances to the pores. This is beneficial to a point, for it enhances purification of the wastewater, but it does slow absorption. Fortunately, the clogging mat will continue to transmit water, albeit at a reduced rate. The flow rate seems to reach an equilibrium value when the system is operated under uniform conditions. Thus, the problem becomes one of managing the infiltrative surface to prevent the clogging mat from becoming excessive by proper design and operation of the system.

Sizing the Infiltrative Surface--

Loading rates--Direct measurement of how the soil will respond to continuous wastewater loading cannot be done practically. Instead, equilibrium flow rates through clogged soils usually must be estimated from a short term soil test. The test most commonly used is the percolation test. It was first developed in 1926 by Henry Ryon in an effort to prevent septic tank system failure by establishing a rational method of soil absorption field design. Ryon plotted the loading rates of systems experiencing problems in the New York area versus percolation rates measured in the soil adjacent to the system (Federick, 1948). An enveloping curve was drawn to include these points. Separate curves were developed for tile fields (trenches and beds) and cesspools. Ryon used these curves to estimate soil absorption field loading rates for septic tank effluent (See Figure B-61). Adoption of this design procedure by the New York State Health Department led to its wide acceptance since few other design criteria existed.

Ryon's design curves were used with varied success for many years throughout the United States. However, after World War II an increasing number of families moved out to the suburban fringes of metropolitan areas onto small lots beyond the reach of sewers. Failures of septic tank systems became a common occurrence. This led to the reevaluation of Ryon's percolation test by the U.S. Public Health Service as part of their broad study of septic tank systems during the years of 1947 to 1953 (Weibel, et al., 1949; Bendixen, et al., 1950; Weibel, et al., 1954).

The Public Health Service investigated 45 tile fields in much the same manner as did Ryon (Bendixen, et al., 1950). Of the systems investigated, 27 had no history of difficulties, 9 had a history of occasional surface seepage while 9 had continuous problems. The results of this study, plotted over Ryon's original curve (Figure B-61) showed the curve to fit fairly well, but the failure rate would be too high if the use of Ryon's curve were to be continued. It was speculated that changes in sewage characteristics since Ryon's original work and climatic factors different from those of New York were influencing the infiltration rates (Weibel, Bendixen and Coulter, 1954). The new data suggested that the equilibrium loading rate was equivalent to a 98% reduction in the percolation rate. This relationship was used in part to develop a new design curve (Figure B-61) (Weibel, Bendixen and Coulter, 1954), which was later published with further modifications (See Table B-16) in the USPHS Manual of Septic Tank Practice (1967).

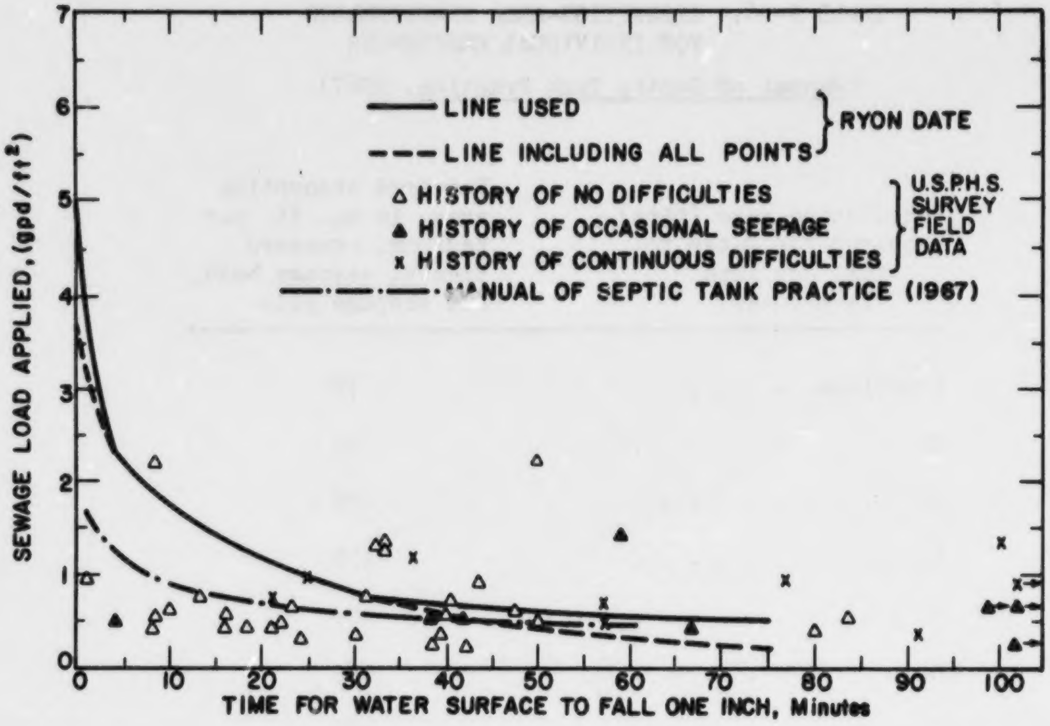

Figure B-61. Relationship of tile field loading rates to percolation test rates (after Bendixen *et al*., 1950)

In addition to decreasing Ryon's recommended maximum loading rates, other significant changes were made. Loading rates were expressed in terms of required absorption area per bedroom served, rather than square feet per capita or gallons per day per square foot. This change required that the potential future use of the home be designed into the system, rather than basing it on present use. Also, it was recommended that soil absorption systems not be permitted in soils with percolation rates slower than 60 minutes/inch and seepage pits not be used where the percolation rates are slower than 30 minutes/inch.

The revised design loadings alleviated the problem to some degree, but many failures still occurred which could not always be correlated to soil type or loading rate. Some of the failures could certainly be attributed to poor construction techniques or lack of septic tank maintenance by the owner, but many others indicated inadequacies of the percolation test itself, which is highly variable. Tests run at the same site by the same technician have been shown to vary as much as 90 percent (Bouma, 1971).

TABLE B-16. ABSORPTION-AREA REQUIREMENTS FOR INDIVIDUAL RESIDENCES

(*Manual of Septic Tank Practice*, 1967)

Percolation rate (time) required for water to fall one inch in minutes)	Required absorption area, in sq. ft. per bedroom, standard trench, seepage beds, and seepage pits
1 or less	70
2	85
3	100
4	115
5	125
10	165
15	190
30	250
45	300
60	330

Modifications of the percolation test have been tried in an attempt to reduce variability but they have met with little success (Weibel, *et al.*, 1949; Bendixen, *et al.*, 1950; Weibel, *et al.*, 1954; Winneberger, *et al.*, 1960; Bouma, *et al.*, 1972). Other attempts have been made to correlate loading rates to specific soil properties, such as the saturated permeability (Healey and Laak, 1974; Bouma, *et al.*, 1972) or soil texture sieve analyses (Norwegian Department of the Environment, 1975), but while these may reduce the variability of the test, interpretation still must rely on an empirical relationship to arrive at an acceptable design loading rate. Saturated hydraulic conductivity tests do not reveal how the soil will conduct wastewater under prolonged loading because the clogging mat restricts liquid movement, with the effect that the soil below the mat is unsaturated. Thus, the flow rate through the soil is governed by the soil's unsaturated hydraulic conductivity which will vary with texture, structure and mineralogy. Soil texture sieve analyses also give limited insight to the percolative capacity of the soil because structure and mineralogy are not taken into account.

With no reasonably simple alternative to determine the equilibrium infiltration rate of soils under wastewater application, the percolation test continues to be favored. However, other information, such as soil borings and soil maps, is often used to increase its reliability. In light of this, Machmeier published an extensive literature review to determine if more recent research and field experience since the publication of the *Manual of Septic Tank Practice* (1967) would suggest modified loading rates (Machmeier, 1975). His recommendations departed little from the *Manual of Septic Tank Practice* (1967), but they represent the most current design loadings (See Table B-17).

If the soil's ability to accept liquid during wastewater application is to be accurately predicted, consideration of unsaturated flow phenomena due to soil clogging mats or compaction is essential. Clogging mats or compacted soil layers of progressively higher resistances will allow progressively lower rates of infiltration through the soil. While a method does not yet exist to measure this rate directly, it can be done indirectly with some accuracy by use of Darcy's Law:

$$Q = KA\frac{dH}{dZ}$$

in which Q = flow rate (cm^3/day); K = hydraulic conductivity (cm/day); A = cross-sectional flow (cm^2); H = gravitational + matric potentials (cm); Z = vertical distance from datum (cm); and dH/dZ = hydraulic head gradient (cm/cm) (Bouma, 1975).

The hydraulic conductivity, which is the one-dimensional flow rate through a unit area under a unit hydraulic gradient, is a reliable measure or any saturated or unsaturated soil to accept and conduct liquid. The crust test was developed for field measurement of K values in terms of the soil moisture potential. Figure B-62 presents the general K-curves developed for the major textural groupings in Wisconsin. These curves relate K to the soil moisture potential. Continued research may result in different

TABLE B-17. COMPARISON OF RECOMMENDED LOADING RATES FOR SOIL ABSORPTION FIELDS

Percolation Rate	Recommended Rates from *Manual of Septic Tank Practice* (1967)				Suggested Loading Rates for Bottom Area (Machmeier, 1975)[1]									
					Beds		Trenches Depth of rock below distribution pipe cm (inches)							
							15	(6)	30	(12)	45	(18)	60	(24)
min/inch	m^2 bedroom	m^2 bedroom	cm/day	(gpd/ft)	cm/day (gpd/ft^2)									
1/2 or less	6.5	(70)	9.5	(2.15)			Not suitable for adequate wastewater treatment							
1/2 - 5	11.6	(125)	5	(1.20)	5	(1.20)	5	(1.20)	6.5	(150)	8	(1.80)	8.5	(2.00)
6 - 15	17.7	(190)	3.5	(0.80)	3.5	(0.80)	3.5	(0.80)	4.5	(1.00)	5	(1.20)	5.5	(1.30)
16 - 30	23.2	(250)	2.5	(0.60)	2.5	(0.60)	2.5	(0.60)	3.5	(0.75)	4	(0.90)	4.5	(1.00)
31 - 45	27.9	(300)	2	(0.50)	2	(0.50)	2	(0.50)	3	(0.65)	3.5	(0.75)	3.5	(0.85)
46 - 60	30.7	(330)	2	(0.45)	2	(0.45)	2	(0.45)	2.5	(0.55)	3	(0.70)	3.5	(0.75)

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[1] Based on 150 gpd/bedroom

groups at a later date. Through the use of tensiometry, the soil moisture potential and gradient can be measured. This technique does not require disruptive removal of soil samples and, therefore, is very suitable for continuous in situ monitoring of moisture conditions in soil surrounding absorption systems. Thus, the moisture potential and its gradient measured in situ can be translated into a flow rate by using Darcy's Law (Bouma, 1975).

This relationship between hydraulic conductivity and soil moisture potential can be used to provide sizing criteria for conventional septic tank systems. Assuming steady infiltration through a barrier of semi-infinite length into soil, the flux through the barrier, Q_b, should equal the flux in the underlying soil, Q_s:

$$Q_b = Q_s \text{ or } K_b \left(\frac{dH}{dZ}\right)_b = K_s \left(\frac{dH}{dZ}\right)_s$$

in which K_b and K_s are the hydraulic conductivities of the barrier and the underlying soil respectively and $\frac{dH}{dZ}$ the hydraulic gradient in both materials (Bouma, 1975). The hydraulic gradient in the soil is approximately unity

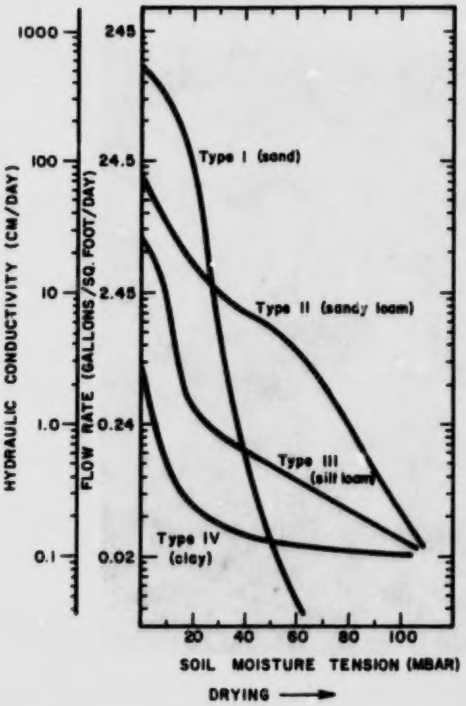

Figure B-62. Hydraulic conductivity (K) of the major soil texture groups in Wisconsin as a function of soil moisture tension measured *in situ* with the crust test procedure (Bouma, 1975) under steady infiltration (Baver, et al., 1972):

$$Q_b \simeq K_s$$

This permits the equilibrium loading rate to be estimated from measured soil moisture potentials under operating systems when K-curves for the underlying soil are available.

Moisture potentials were measured under several ponded conventional septic tank-soil absorption systems to determine equilibrium flow rates through clogging mats in different soils (Bouma, 1975; Bouma, et al., 1972; Bouma, et al., 1975a; Magdoff and Bouma, 1974; Walker, et al., 1973a). The tensiometers consisted of 5 cm long porous cups attached to 2 cm diameter plastic tubing. These were connected to mercury manometers with fine tubing (See Figure B-63). Small excavations were made adjacent to ponded systems and the tensiometers were installed at different points in the soil below and to the side of the systems. Measured potentials were used to estimate infiltration rates into the soil through the bottom and sidewalls of the system, using the appropriate K-curve. Where direct measurement of moisture potentials was not possible, soil samples were taken next to the bed to obtain moisture contents, which were translated into moisture

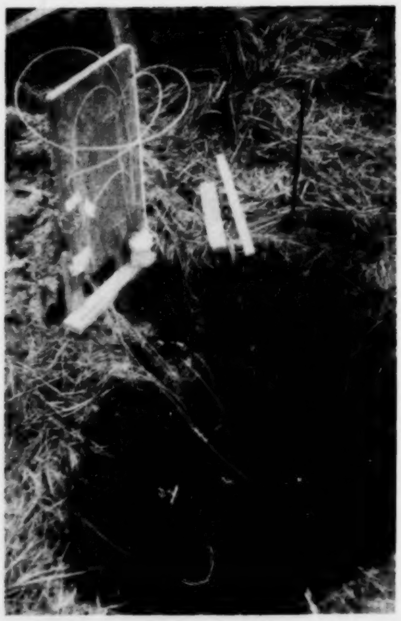

Figure B-63. *In situ* measurement of soil moisture tensions in soil adjacent to subsurface seepage systems. Porous cups inserted in the soil are connected through fine tubing with a mercury reservoir. Moisture tensions are derived from the mercury rise along a calibrated scale (Bouma, et al., 1972).

potentials using moisture retention curves (Bouma, et al., 1972). The moisture potentials were used to obtain K-values from K-curves derived from the particular soil found at each site, rather than referring to the general curves shown in Figure B-62.

This technique is valid if the water table is deep and the flow in the underlying soil is vertical and one-dimensional under unit gradient. These conditions are closely approximated immediately under the clogging mat some distance from the edge of the system. However, in most cases it was necessary to make measurements 5 to 10 cm below the mat and near the edges of the absorption areas because of difficulties in excavation. In this location, flow might not be one-dimensional but diverging with the effect that the gradient would be greater than unity. Thus, in equating K to Q_b as in the previous equation, the estimated flow rate would be less than the actual flow rate. This results in a conservative estimation of the infiltration rate.

Characteristics and performance indicators based on in situ monitoring data for 12 septic tank-soil absorption systems are presented in Table B-18. All systems, except as noted, used 10-cm perforated pipe to distribute the septic tank effluent in the absorption field by gravity. Thirty centimeters of gravel lay under the pipe.

Hydraulic characteristics of the soil in terms of K-curves could be broadly classified into four groups as shown in Figure B-62. Results of the studies presented in Table B-18 are discussed for each group.

Conductivity Type I (sands)--Eleven conventional systems were investigated in this soils group. All those over 9 months of age were found to be ponded due to clogging of the infiltrative surface (Bouma, et al., 1972). Four were selected for additional monitoring of the moisture tensions below the clogging mat. Results of this monitoring showed that moisture tensions and associated flow rates in soil surrounding clogged trenches or beds were not very different for the different systems, despite their differences in system age. This would seem to indicate that a mature clogging mat is established early in the system's life and that flow rates through the mat change little as the system ages. The results also show that clogged sands accept significant quantities of septic tank effluent through both bottom and sidewall surfaces. However, the hypothesis that sidewalls are more effective than bottom areas as infiltrative surfaces (McGauhey and Winneberger, 1965) is not supported by these data. The choice of which surface should be favored in design may depend on the local climate.

Based on these data, 5 cm/day (1.2 gpd/ft^2) is recommended as a maximum loading rate of the bottom area for systems with 30-cm (12 inch) sidewalls constructed in sands (Bouma, 1975). This rate compares closely with that commonly used when assuming 150 gpd/bedroom (See Table B-17). It is further recommended that the effluent be applied uniformly over the entire bottom infiltrative surface in four or more daily doses, particularly during system start-up, if bacterial and viral contamination of a shallow water table is a concern. The uniform application in small volumes will insure unsaturated conditions in the sand necessary for good purification.

TABLE B-18. CHARACTERISTICS AND ESTIMATED LOADING RATES OF OPERATING CONVENTIONAL SOIL ABSORPTION SYSTEMS IN WISCONSIN

	System No.	System Age	System Type	Evaluation of System Performance	Soil Texture	Soil Type and Horizon	Percolation Rate (min/inch)	Depth of Ponding (cm)	Soil Tension (mbar) Bottom	Soil Tension (mbar) Sidewall	Estimated Flow Rate cm/day (gpd/ft²) Bottom		Estimated Flow Rate cm/day (gpd/ft²) Sidewall		References
		(Years)					min/inch	cm	mbar	mbar	cm/day	(gpd/ft²)	cm/day	(gpd/ft²)	
Type I	1	12	trench	satisfactory	coarse sand	Plainfield ls. C horizon	2	30	20-23	23-35	7.5	(1.8)	1.7	(0.4)	Bouma et al. 1972
	2	5	bed (resort)	satisfactory	coarse sand	Plainfield ls. C horizon	2	25	21-25	23	5.8	(1.4)	5.8	(1.4)	Bouma et al. 1972
	3	7	trench	some seepage	coarse sand	Rubicon ls. C horizon	2	5	23-27	21	6.6	(1.6)	9.2	(2.2)	Bouma et al. 1972
	4	2	trench	satisfactory	coarse sand	Vilas ls. C horizon	1	30	23-25	28	7.5	(1.8)	4.6	(1.1)	Bouma et al. 1972
Type II	5	11	trench	satisfactory	gravelly sandy loam	Saybrook sil. IIC horizon	6	25	80	60	0.4	(0.09)	[illegible]	(0.2)	Bouma et al. 1972
	(5a)	1	(dosing)	satisfactory				0	60	-	0.8	(0.2)	-		Bouma et al. 1972
	6	6	trench	failing	gravelly sandy loam	Batavia sil. IIC horizon	6	150	120	100	0.03	(0.01)	0.04	(0.01)	Bouma, 1975
	7	2	bed	satisfactory	gravelly sandy loam	Saybrook sil. IIC horizon	30	1	65	-	1.9	(0.45)	-		Bouma, 1975
Type III	8	3	bed	failing	silt loam	Batavia sil. B3 horizon	30	15	34	37	0.7	(0.16)	0.6	(0.14)	Bouma, 1975
	9	2	bed	failing	silty clay loam	Plano sil. B3 horizon	45	75	-	22	-		0.7	(0.16)	Bouma et al. 1975b
	10	3	bed	failing	silt loam	topsoil fill over Plano sil.	55	40	20	20	1.0	(0.23)	1.0	(0.23)	Bouma et al. 1975b
	11	1	bed (dosing)	satisfactory	silt loam	Decorra sil.	50	0	-	-	>0.8	(>0.2)[1]	-		Bouma et al. 1975a
	12	1.5	bed (restaurant)	poor	silt loam	Kegonsa sil.	30	30	34	37	0.6	(0.15)	0.6	(0.15)	Bouma et al. 1975a
	13	0.5	bed (dosing)	satisfactory	silt loam	Decorra sil.	40	0	-	-	>0.75	(>0.18)[1]	-		Bouma et al. 1975a
	14	1	trench (dosing)	satisfactory	silt loam	Tama sil.	30	0	-	-	>1.75	(>0.42)[1]	-		Bouma et al. 1975a
	15	3	trench (press. dist.)	satisfactory	silt loam	Batavia sil. B3 horizon	30	0	-	-	7.2	(1.8)[1]	-		Bouma et al. 1975a
	16	3	trench (press. dist.)	satisfactory	silt loam	Batavia sil. B3 horizon	30	0	-	-	6.8	(1.7)[1]	-		Bouma et al. 1975a
Type IV	17	1	trench	failing	clay	Kibbing sil. B2 horizon	900	15	6	6	0.17	(0.04)	0.17	(0.04)	Bouma et al. 1972
	18	2	trench (press. dist.)	satisfactory	silty clay	Briggsville l.	90	20	15	20	0.75	(0.18)	0.62	(0.15)	Bouma, 1975

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[1] Calculated from estimated wastewater volume.

Conductivity Type II (sandy loams; loams)--Soils of this type have rapid percolation rates initially but they have a tendency to clog quite severely. This may be due to their particular pore size distribution and structural instability. Relatively low clay contents do not allow significant swelling and shrinking of the soil necessary to form structural units or peds with associated interpedal cracks. Tubular worm and root channels are formed, but they tend to be more unstable and much less permanent than those formed in clayey soils. Thus, the packing pores between particles, which are much finer than in the sands, are the principal voids through which the water moves (Bouma and Anderson, 1973). The finer pores may result in greater accumulation of solids at the infiltrative surface and the development of anaerobic conditions in the clogged layer due to the reduced air diffusion compared to sandy soils (Magdoff, et al., 1974a).

Seven operating systems were investigated in these soils, and all were ponded with septic tank effluent. Systems 5, 6 and 7 were selected for more detailed study (See Table B-18). The moisture tensions measured below the bottoms of the systems increased with increased ponding depth within the systems indicating that the clogging mat resistance increased with ponding depth. The estimated rates through the bottom areas varied from 0.4 cm/day (0.09 gpd/ft^2) to 1.9 cm/day (0.45 gpd/ft^2). The estimated rates through the sidewalls were similar.

These data indicate that relying on the percolation rate alone for sizing the system is not sufficient, for soils of this type clog much more easily than the percolation rate would seem to indicate. To maintain reasonable infiltration rates,the data further suggest that ponding levels within the system should be kept to a minimum.

This finding is contrary to what has been reported by others (Healey and Laak, 1974; Kropf, et al., 1975). Kropf, et al., (1975, 1977) found that during 70-day laboratory column studies, increasing the ponding depth over the infiltrative surface increased the amount of wastewater absorbed, but not as great as would be predicted by Darcy's Law. Increasing the ponding depth in finer textured soils was not as effective (Kropf, et al., 1975). This fact was attributed to the nature of the stresses placed on the mat which would have more of a disruptive effect in sands because of the poor supportive matrix due to the large pore size. If the tests had been run longer, however, the increased infiltration rate may have been found to be temporary. This transient increase is suggested by the more recently published data (Kropf, et al., 1977). If true, these data would confirm that increasing ponding depths lead to a corresponding increase in clogging mat resistance.

To reduce ponding levels, intermittent periods of aeration between applications should be provided to allow aerobic decomposition of the clogging mat. To test this hypothesis,one trench of System 5 was drained and allowed to dry before wastewater was reapplied in once per day dosings (System 5A, Table B-18). After several months of operation in this mode, the moisture tensions below the clogging mat had dropped from 80 mbar to 60 mbar indicating the mat was passing more liquid. When the operation returned to continuous application, the moisture tensions again increased to 80 mbar (Bouma, et al., 1972).

Absorption fields designed for bottom area loadings of 3 cm/day (0.7 gpd/ft^2) with 30-cm (12-in) sidewalls have functioned well in Wisconsin. Trenches are preferred to beds, with once daily dosing recommended if this rate is used (Bouma, 1975). This rate is somewhat lower than design rates used elsewhere (See Table B-17).

Conductivity Type III (silt loams, some silty clay loams)--Although these soils are more finely textured than either Type I or Type II soils, their more strongly structured nature can maintain relatively high infiltration rates if the system is constructed and managed properly. Nine systems were investigated (Systems 8-16, Table B-18). Four of these systems, all bed designs, were failing or about to fail. The cause of the failures were traced to construction problems (Bouma, 1975; Bouma, et al., 1975a). Construction of beds often involves several passes over the infiltrative surface by machinery while excavating and placing of the rock. This practice can result in severe compaction and puddling if the soil is wet, because these finer textured soils exhibit a plastic consistancy over a wide range of moisture contents, which occur naturally in the field (Bouma, 1975). Observations made at the installations indicated that excavating equipment had been driven over the bottom areas of the beds during construction. The presence of a compacted layer was confirmed by moisture tension measurements taken below the beds. These indicated a restricting layer with a resistance reasonably close to resistances through layers of manually puddled fine silty soil materials used in the original version of the crust-test procedure (Bouma, et al., 1971; Bouma, et al., 1975a). The other five systems studied were functioning satisfactorily and did not contain ponded effluent. Two were beds both of which were dosed. Samples taken of the soil from the bottom of the systems showed well exposed soil structure with open planar voids between peds as well as worm and root channels. The exposure of these larger pores explains the lack of ponding. For example, one tubular root channel with a diameter of only 2 mm (0.008 in.) can conduct 285 L/day (75 gal/day) at a hydraulic gradient of 1 cm/cm (Bouma and Anderson, 1973). This points to the importance of construction practices which minimize damage to the structure of the soil.

The results from these investigations also demonstrate the advantages of dosing. Systems 11 and 13 through 16 were all dosed by pumping effluent through the distribution piping. Conventional 10-cm (4-inch) pipe, perforated near the inverts, was used in each of systems 11, 13 and 14. System 15 used 10-cm (4-inch) pipe perforated at the crown of the pipe, requiring the pipe to fill before discharging liquid, and System 16 used small-diameter pipe with small orifices. These designs were used to achieve more uniform distribution. None of these systems were ponded, indicating that infiltration rates were greater than the application rate. Since ponding did not occur, it was necessary to estimate infiltration rates from the daily volume of wastewater discharged (Bouma, et al., 1975a). Bottom areas were the only infiltrative surface in these instances. The reported infiltration rates in Table B- for Systems 11, 13 and 14 do not reflect the true rate, but merely its minimum, because the loading could not be changed. To provide more flexibility in loading, Systems 15 and 16 were constructed in a deep fine silty loess deposit overlying a calcareous permeable sandy loam glacial till as experimental systems (Bouma, et al., 1975a). By dosing once daily, uniformly over the surface, infiltration rates of 7.2 cm/day (1.8 gpd/ft^2) and 6.8 cm/day (1.7 gpd/ft^2) were realized after more than 2 years of operation. These rates are nearly three times those recommended in most areas (See Table B-17). Excavations into the trenches revealed recent worm and other fauna activity which left large vertical channels through the infiltrative surface. Dosing and uniform distribution, with drying periods under aerobic conditions between applications, may stimulate this activity, as organisms seek the nutrients deposited at the infiltrative surface. This would seem to suggest that while good construction practices are necessary to expose an open infiltrative surface, periodic application of effluent is essential to keeping the surface open. This is in opposition to the conclusion reached by Healey and Laak (1974) and Kropf, et al., (1975, 1977) who worked only with Type I and Type II soils.

While the data are not conclusive, they suggest that maximum permissible loading rates would vary according to the method of distribution employed. If once daily dosing were employed, maximum rates of 5 cm/day (1.2 gpd/ft^2) might be acceptable based on bottom area only (Bouma, 1975). Uniform distribution would be crucial in this case to maintain unsaturated flow so that deep penetration of pollutants through the large exposed pores will not occur. If conventional gravity trickle distribution is used, the conventional loading rate of 2 cm/day (0.5 gpd/ft^2) should not be exceeded. In both cases, shallow trench designs 45 to 60 cm (18 to 24 in) deep are preferred because the upper soil horizons are usually more porous and less subject to damage during construction. Shallow systems also enhance evapotranspiration.

Conductivity Type IV (clays, some silty clay loams)--Low conductivities in these soils at saturation drop strongly in the 0 to 20 mbar tension range due to the emptying of the interpedal voids and tubular channels as in Type III soils (See Figure B-62). However, lower K_{sat} values indicate the lack of many large pores. Thus, the soil itself, rather than the clogging mat, becomes the dominant controlling factor (Bouma, 1975; Healey and Laak, 1974).

Only two systems of this type were investigated. System 17 was observed to be slightly smeared and compacted during construction while System 18 was installed under ideal conditions. Effluent from an aerobic treatment unit was dosed into System 12 using pressure distribution.

Because soils of this type are severely limited, it may be more crucial to maintain an open infiltrative surface to utilize the large interpedal cracks and tubular channels. Dosing frequencies of once per day or longer may promote soil fauna activity between dosings to maintain an open surface (Bouma, et al., 1975a). If conventional gravity distribution is used, loading rates of 1 cm/day (0.2 gpd/ft^2) based on the bottom area only would seem to be acceptable, assuming 33 percent of the flow would be through the sidewall (Bouma, 1975). If expandable clays are present, a lower rate should be used.

For a summary of recommended loading rates, see Table B-19.

TABLE B-19. RECOMMENDED MAXIMUM LOADING RATES FOR SEPTIC TANK SOIL ABSORPTION FIELDS BASED ON *IN SITU* MEASUREMENTS[1] (After Bouma, 1975)

Conductivity Type	Soil Texture	Loading Rate[2] cm/day (gpd/ft^2)	Operating Conditions
I	Sand	5 (1.2)	4 doses/day Uniform Distribution Trenches or Beds
II	Sandy Loams	3 (0.7)	1 dose/day Uniform Distribution Trenches Preferred
	Loams	2 (0.5)	Conventional Distribution Shallow Trenches
III	Silt Loams Some Silty Clay Loams	5 (1.2)[3]	1 dose/day Uniform Distribution Shallow Trenches Only
IV	Clays	1 (0.2)[3]	1 dose/day Uniform Distribution Desirable Shallow Trenches Only

[1] Assumes that the high water table is > 90 cm (3 ft) below the infiltrative surface.

[2] Bottom area only.

[3] Should not be applied to soils with expandable clays.

Bottom versus sidewall area--Both the horizontal bottom area and vertical sidewalls of a subsurface soil absorption system can act as infiltrative surfaces for wastewater absorption. When a conventional gravity system is first put into operation, the bottom area is the dominant infiltrative surface. However, after a period of wastewater application, this surface can become clogged sufficiently to pond liquid above it, at which time the sidewalls become infiltrative surfaces. Because the gradients and resistances of the clogging mats at the two surfaces are rarely the same, the infiltration rates will be different. Which surface would have the greatest infiltration rate will depend on a number of factors. Vertical and horizontal hydraulic conductivities and gradients in the soil, clogging mat resistances, and soil moisture contents of the surrounding soil are factors that will effect the direction and rate of liquid movement through the soil. Thus, the more significant infiltrative surface may vary between sites. The objective in design is to maximize the area of the surface expected to have the highest flow rate.

Based on investigations done at the University of California in Berkeley, McGauhey and Winneberger (1965) have reported that the sidewall is ". . . by far the most effective infiltrative surface." They concluded from various studies with packed lysimeters (Winneberger, Saad and McGauhey, 1961) and columns (Winneberger, Menar and McGauhey, 1962) of Hanford fine sandy loam, Yolo sandy loam, and Oakley sand that (1) suspended solids in the effluent do not contribute to sidewall clogging, (2) rising and falling liquid levels within the system allow alternate loading and resting of the surface while the bottom is often continuously inundated, and (3) sloughing of the clogging mat can occur during resting periods. Therefore, they recommend that subsurface soil absorption systems should provide a maximum of sidewall surface per unit length of trench and a minimum of bottom surface (McGauhey and Winneberger, 1965).

The Manual of Septic Tank Practice (1967) recognizes the contribution by the sidewall but recommends the bottom area as the principal infiltrative surface. A statistical allowance for the sidewall is included in the recommended bottom area per bedroom, assuming a 15-cm (6-in) vertical sidewall (See Table B-17). If deep trenches are used, the total bottom area of the trench can be reduced by a factor determined by the relationship:

Percent of length of Standard Trench

$$[15\text{-cm (6-in) sidewall}] = \frac{w + 2}{w + 1 + 2d} \times 100$$

where w = the width of the trench and d = the depth of the gravel below the distribution pipe. While this gives credit for sidewall absorption, it assumes that the infiltration rate is less than that of the bottom. No allowance is made for deep beds.

The extent to which the sidewall becomes an infiltrative surface would depend upon the prevailing hydraulic gradient and clogging mat resistance. The hydraulic gradient is largely determined by the soil type and soil wetness surrounding the system. At the bottom surface, the gravitational potential, the pressure potential of the ponded water above, and the matric potential of the soil below each contribute to the total potential of the liquid, while at the sidewall the gravitational potential is zero since it operates only vertically and the pressure potential of the ponded water diminishes to zero as the liquid surface is approached. The lower hydraulic gradient across the sidewall can be offset if the clogging mat is less resistant. This would be expected since the rising and falling liquid levels, in effect, alternately dose and rest the sidewall as pointed out by McGauhey and Winneberger (1965). Monitoring of operating absorption fields by Bouma, et al. (1972), seems to confirm this fact. Estimated flow rates through the sidewalls and bottom areas were not significantly different in most cases though the hydraulic gradients did vary. However, in temperate climates, frequent rainfall, particularly in the spring and fall, may reduce the matric potential at the sidewall to low levels due to percolating precipitation. During such times, the horizontal gradient could be reduced to a very low level with the effect that the bottom surface becomes the only reliable infiltrative surface. Healey and Laak (1974) recommend that in temperate zones subsurface absorption systems be designed to function under gravity potential only, because of the problems during wet portions of the year.

To increase the hydraulic gradient across the sidewall, deep narrow trenches could be constructed as recommended by McGauhey and Winneberger (1965). However, this would diminish the advantages of shallow trenches which enhance evapotranspiration and avoid construction in the deeper soil horizons where puddling and compaction are often more likely. It might be concluded that in humid regions systems should be designed on bottom area while maximizing the sidewall by utilizing trenches rather than beds. In more dry regions, with deep permeable soils, the sidewall area could be maximized at the expense of the bottom area.

Shallow versus deep absorption systems--Shallow soil absorption systems offer several potential advantages over deep systems: 1) the upper soil horizons are usually more permeable than the deeper subsoil because of clay migration to the deeper horizons and because of greater plant and soil fauna activity; 2) evapotranspiration is greater; 3) the upper soil dries more quickly than the subsoil so construction can proceed over longer periods of the year with less smearing, puddling and compaction; and 4) less excavation is necessary, reducing the cost. Deep systems are desirable when the sidewall is determined to be the better infiltrative surface, more permeable soil exists with depth, or freezing is a danger during cold periods of the year.

Freezing of shallow absorption systems does not seem to be a problem, even when frost penetration is quite deep. Weibel, et al., (1949) reviewed the literature and made contacts with health authorities and plumbers in the northern states to determine if failures of shallow systems due to freezing were frequent. They concluded that carefully constructed shallow systems 45 to 60 cm (18 to 24 in) in depth would not freeze even in areas where frost penetration reaches 1.5 m (60 in), if the tile lines were gravel packed, insulated under driveways or other surfaces usually cleared of snow, and kept in reasonably continuous operation.

This was confirmed by a field study in which three adjacent trenches were monitored for temperature. The trenches where each 75 cm (29.5 in) deep, with 33 cm (13 in) of gravel placed below the distribution pipe. Four centimeters (1.6 in) of gravel covered the pipe. Topsoil was filled over the gravel to a depth of 38 cm (15 in) which brought the top of the system 10 cm (4 in) above the original surface. Thermocouples were placed at the top of the gravel, within the pipe, and in the natural soil beside the trench 15 cm (6 in) above the trench bottom and 10 cm (4 in) below the trench bottom. Only Trench #2 was loaded by dosing once daily. The other two trenches stood idle.

Temperatures have been monitored weekly since July, 1972. The winter of 1976-77 was the most severe. Average daily ambient air temperature remained below 0°C (32°F) from late in November, 1976 to early February, 1977. During much of this period, the average temperatures were between

-18° and -26° C (0° to 15°F) (See Figure B-64). The temperatures in the pipe and below the bottom of the unloaded trench dropped below freezing for over a 6-week period as the frost penetrated the ground. However, in the trench which was dosed daily with septic tank effluent temperatures remained above freezing throughout this period.

Trench versus bed design--Though the seepage bed is often more attractive than seepage trenches in terms of total land area requirements, cost, and ease of construction for the same bottom area, it is less desirable in terms of maintaining the infiltrative and percolative capacity of the soil. This is particularly true in soils with significant clay contents (>25% by weight). The principal advantages of trenches over a bed are: 1) more infiltrative surface is provided for the same bottom area and 2) less damage to the bottom infiltrative surface occurs due to compaction, puddling and smearing during construction.

For the identical bottom areas, trench designs of absorption fields can provide more than eight times the sidewall area. This can be of benefit in preventing failure through clogging. In humid climates there may be portions of the year that the sidewall looses much of its effectiveness for absorption, which necessitates designing the system to function on bottom area only. However, it is recognized that the sidewall is beneficial, and it is certainly recommended to maximize it in any system (Bouma, 1975; McGauhey and Winneberger, 1965).

In addition, the seepage bed design can cause severe damage to the natural soil structure during installation. This is a particular concern in clayey soils. Rapid absorption of liquid by the soil depends on a suitable soil structure being maintained (Bouma, 1975; Bouma, et al., 1975a). When mechanical forces are applied to moist or wet soil, the structure is partially or completely destroyed because clay particles in the soil are able to slip relative to one another. This movement, referred to as compaction, puddling or smearing, closes the larger pores between soil aggregates and those made by roots, or burrowing soil fauna.

To construct a seepage bed, it is common practice to first scrape off the topsoil using a front end loader and then return with a backhoe for digging to final grade in an attempt to leave a fresh soil surface. However, these two operations may require several passes over the bed area by the construction machinery, often with heavy loads. When digging is complete, trucks may be backed into the bed to unload aggregate, which is spread over the bottom of the bed by machinery. After the distribution piping is laid, additional gravel is placed over the pipe and covered with soil. By the time the bed is completed, the soil structure may be destroyed.

This problem is further compounded when soil conditions are wet. A busy contractor is unable to always schedule his work when the soil is dry, so construction often proceeds when conditions are marginal at best. The trench design reduces the severity of these problems because the construction machinery is able to straddle the trench so that the future infiltrative surface is never driven upon.

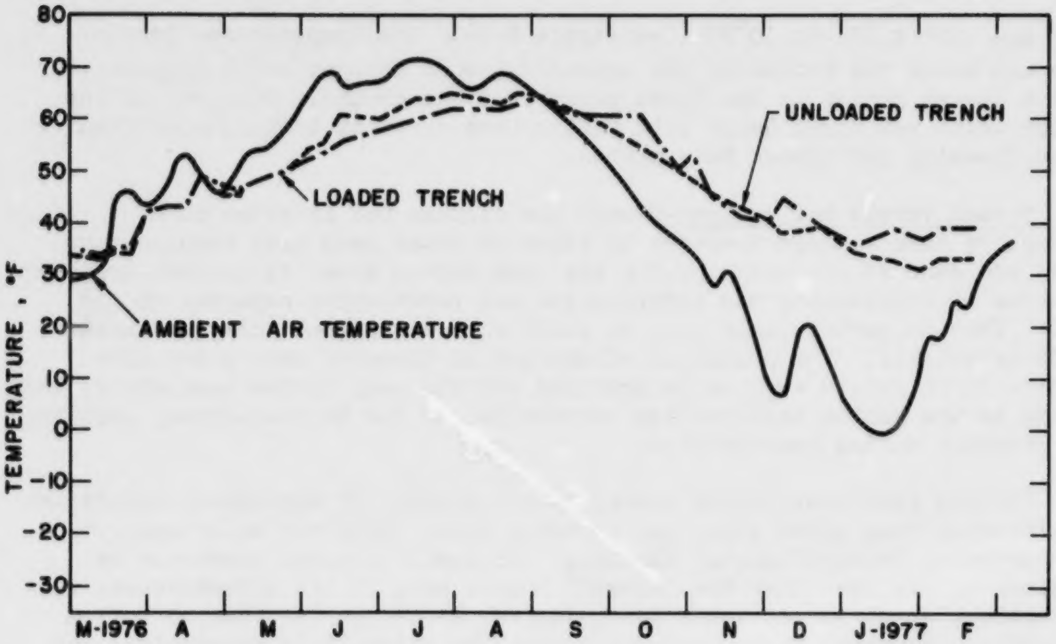

Figure B-64. Temperatures recorded in a loaded and unloaded trench during 1976-1977.

Unfortunately, many state and local codes favor the construction of beds over trenches. The codes usually follow the recommendations of the Manual of Septic Tank Practice (1967), which limits trenches to 1.5 m (5 ft) widths with 1.8 m (6 ft) separations between sidewalls. Thus, a 100 m^2 (1076 ft^2) absorption bed can be laid out in a 8 m x 12.5 m (26 ft x 38 ft) rectangular area while a trench system would require a 6.6 m x 33.3 m (22 ft x 110 ft) area assuming 1 m (3.3 ft) trench widths. These larger areas required by trenches are often undesirable. In addition, trench systems cost up to 25% more in Wisconsin because additional time is required for construction.

To encourage the use of trench systems, codes could be changed. A reasonable approach might be to require more bottom area for beds than trenches for the same size household. Two methods might be used: 1) give credit for sidewall area, thereby reducing the bottom area required for trenches, or 2) increase the bottom area now required for beds in proportion to the amount of sidewall area lost by not using the trench design.

A trench or bed can be represented by a rectangle with sides of x and y. The bottom area becomes:

$$A_B = xy$$

If A_B is set constant, x and y can vary. If x is the width, when it varies between 0.3 m to 1.5 m (1 ft to 5 ft) the system is defined as a trench. If x is greater than 1.5 m (5 ft) then the system is defined as a bed.

If it is assumed that 60 cm (12 in) of gravel is laid below the inlet of the distribution pipe, the sidewall area is:

$$A_S = 2x + 2y$$

Therefore, the sidewall area, A_S, is a minimum when $x = y$ (a square bed), and a maximum when x approaches 0.

A useful means of comparing sidewall areas for different shape and size of beds is a ratio of sidewall area to bottom area:

$$R = \frac{2x + 2y}{xy}$$

Substituting $y = \frac{A_B}{x}$:

$$R = \frac{2x^2 + 2A_B}{A_B x}$$

This analysis indicates that trenches 1.5 m (5 ft) wide can provide 25% more sidewall area than beds for 30 cm (12 in) deep systems. Since bed designs are most unfavorable in finer textured soils, a 25% increase in bottom area required for beds is reasonable. This would have the effect of reducing the advantages of cost and land area required which beds offer over trenches.

Liquid distribution--

Dosing and uniform application of wastewater effluent over the infiltrative surface may be critical to the proper functioning and long term life of the soil absorption system. Localized overloading due to continuous inundation and poor distribution may result in inadequate purification of the effluent in very permeable soils and accelerated clogging in all soils (Bouma, 1975; Bouma, et al., 1972; Robeck, et al., 1964; McGauhey and Winneberger, 1964).

In conventional subsurface soil absorption systems, distribution of the effluent within the field is usually provided by perforated drain tile laid below the elevation of the septic tank outlet to permit gravity flow of the liquid. The pipe is commonly 10 cm (4 in) in diameter and perforated with two rows of 1 cm (3/8 in) diameter holes spaced 7.5 cm (3 in) apart. It is laid level or on a slope of 0.167 to 0.333 percent such that the rows of holes are positioned downward 45° either side of the vertical center line of the pipe. One pipe is used per trench, but in the case of a bed, two or more pipes may be installed with a spacing of 90 cm to 180 cm (3 ft to 6 ft) between centers. In a bed or multi-trench system the pipes are usually interconnected by a common solid wall header pipe or through a distribution box (Manual of Septic Tank Practice, 1967).

This method of effluent application provides very poor distribution. McGauhey and Winneberger (1964) and Bouma, et al., (1972) observed a phenomenon of "creeping failure" within trenches and beds. It was attributed to the discharge of effluent out the holes in the pipe nearest the inlet to the absorption field. The soil below this point becomes overloaded, receiving a more or less continuous trickle of effluent. Biological clogging soon occurs which reduces the rate of infiltration below the rate which the effluent is discharged, forcing the effluent to flow along the bottom of the system until it encounters unclogged soil. This process continues until the whole system is ponded (See Figure B-65). Failure does not necessarily occur at this time if the system is sized to account for the reduced rate of infiltration. However, the initial overloading of the unclogged soil may result in groundwater contamination in sands while continuous inundation which ultimately occurs may cause severe clogging, leading to surface seepage in some finer textured soils.

In an attempt to improve distribution, several full-scale distribution systems were evaluated in the laboratory (Converse, 1974). A gravel trench was constructed such that water passing through the gravel from each 45-cm (18 in) segment could be measured. Both gravity flow and pumped discharge systems were tested.

The conventional 10 cm (4 in) diameter bituminous fiber pipe with two parallel rows of holes located downward provided very poor distribution. Figure B-68 shows a typical distribution pattern for a 14.6 m (48 ft) length of pipe laid on a 0.215 percent slope when 57 L (15 gal) of water were allowed to flow by gravity. The liquid distribution was concentrated at the head and end of the pipe with little or no flow between. The first holes, as well as the holes with the lowest elevation, received the greatest proportion of flow. When water was pumped into the pipe at the rate of 48 L/m (13 gpm), 97 percent of the liquid was distributed over 63 percent of

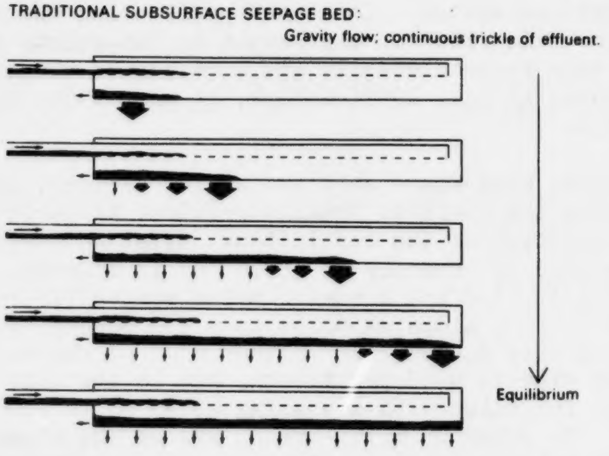
TRADITIONAL SUBSURFACE SEEPAGE BED:
Gravity flow; continuous trickle of effluent.
Equilibrium

Figure B-65. Progressive clogging of the infiltrative surfaces in subsurface seepage beds with gravity distribution characterized by continuous trickle flow (Bouma, et al., 1972) the bed (See Figure B-67). A lesser slope resulted in less than 45 percent of the bed receiving effluent.

The trials were repeated using a single row of holes located in the crown of the pipe. This configuration improved distribution. The most important factors affecting distribution were the slope of the pipe, variation in hole elevation, flow rate and pumping time. Gravity flow was not as effectual (Figure B-68). The results indicated that the pipe should be laid level and a high flow rate provided with few holes in the pipe (See Figure B-69). A minimum flow rate of 95 L/m (25 gpm) for no less than 2.5 min is recommended for a pipe with hole spacing of 90 cm (3 ft) (Converse, 1974).

The results with inverted pipe (holes at the crown) configuration indicate that better distribution is achieved when the pipe is under pressure. When the number of holes are decreased and the flow rate increased, more uniform distribution results. This suggests that the best method of distributing liquid uniformly over a large area would be to utilize a pressure network.

Pressure networks have been used for years in fixed nozzle trickling filter units to uniformly apply wastewater over the filter media. The objective in design is to balance the headlosses to all parts of the network. This is done by maintaining one to two psi of pressure within the perforated laterals at their terminal ends and sizing the perforations and pipe chambers such that the headlosses incurred in delivering the water to the holes does not exceed 10 to 15 percent at the in-line pressure. The pump or siphon used to pressurize the network is sized to supply the necessary flow against the network losses plus the elevation head and friction losses incurred in delivering the liquid to the network (Otis, et al., 1977).

The pressure networks were designed and constructed for evaluation. The networks had single manifolds with 4 to 6 perforated laterals (See Figure B-70a and B-70b). The perforated pipe was 2.5 cm (1 in) diameter, with 0.52 cm (13/64 in) or 0.64 (1/4 in) diameter holes spaced every 75 cm (30 in) (Converse, 1974). Distribution was improved through these networks, though irregularities in the holes due to drilling caused some fluctuations (See Figure B-71a and B-71b). Since the networks are laid with the holes at the pipe inverts allowing the system to drain between dosings, dosing volumes should be large enough to fill the network within 10 percent of the total dosing time.

Six pressure networks were installed at private homes to evaluate their performance under field conditions (Converse, et al., 1975a). After two years of operation, the principal problem encountered was in pump sizing. One-third horsepower submersible pumps were used but were not always sufficiently large to achieve adequate pressure throughout the system. Plugging of the lines occurred in only one system, which was found to be due to improper installation (Otis, et al. 1974).

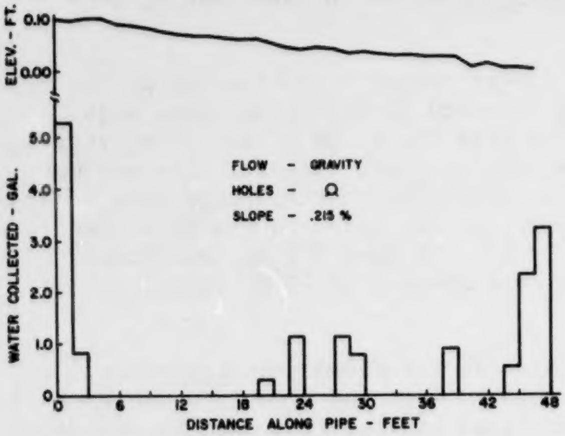

Figure B-66. Gravity distribution of 15 gal. of water from a 4-in perforated bituminous pipe (Converse, 1974).

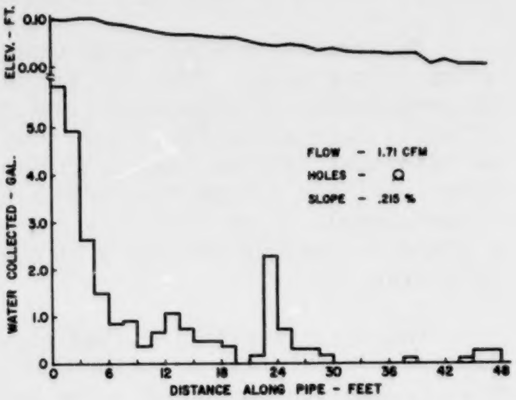

Figure B-67. Pumped distribution of water along a 4-in perforated pipe (Converse, 1974).

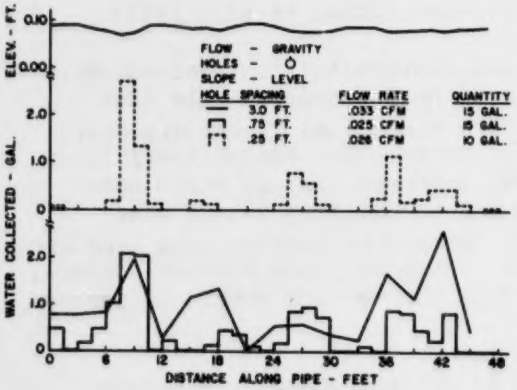

Figure B-68. Gravity distribution of water from a level 4-in perforated bituminous pipe with one row of holes at the crown of the pipe (Converse, 1974)

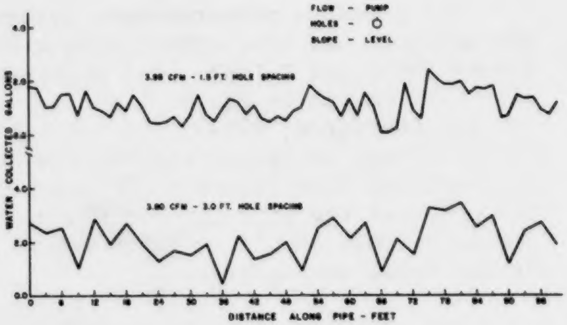

Figure B-69. Pumped distribution for a level 4-in perforated bituminous pipe 100 ft long with one row of holes at the crown of the pipe (Converse, 1974).

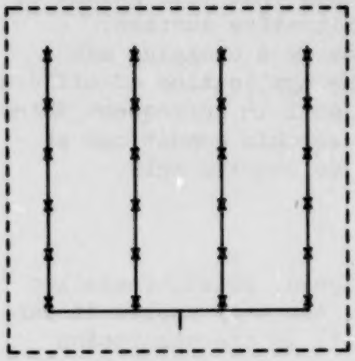

Figure B-70a. The top view of a bed system consisting of a 2.5-cm (1-in) PVC manifold and four laterals, each with six 0.5-cm (13/64-in) holes spaced 75-cm (30-in) apart. The laterals are 4 m (13.5-ft) long and spaced 1.4 m (4.5-ft) apart. The 5.3-m (17.5-ft) square dashed area represents the seepage bed perimeter (Converse, 1974).

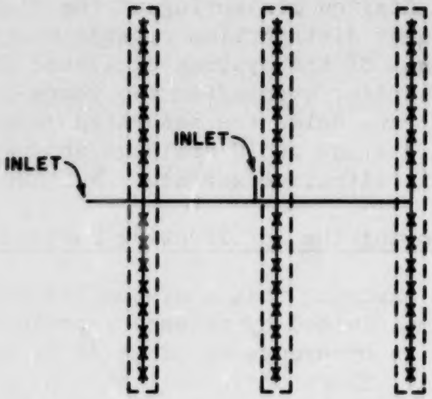

Figure B-70b. The top view of a trench system consisting of a 7.5-cm (3-in) PVC manifold with six 2.5-cm (1-in) PVC laterals. Each lateral has eight 0.6-cm (1/4-in) holes spaced 75-cm (30-in) apart. The inlet is on the end or center. Each 6.1-m (20-ft) lateral is spaced 4.6-m (15-ft) apart. The three 0.9-m (3-ft) dashed areas represent the trench perimeters (Converse, 1974).

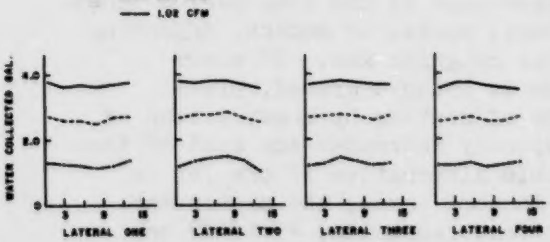

Figure B-71a. Distribution for three flow rates at the bed network (Converse, 1974).

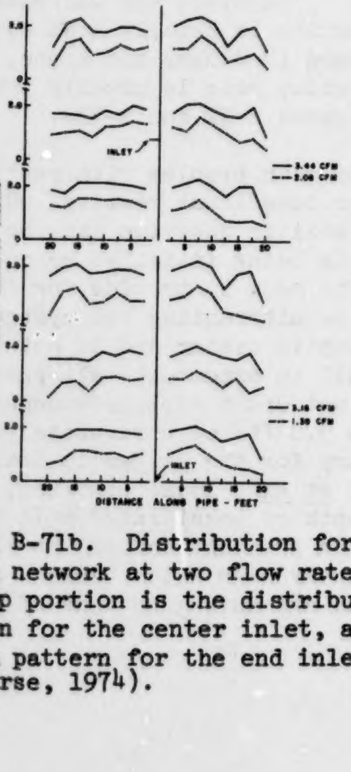

Figure B-71b. Distribution for the trench network at two flow rates. The top portion is the distribution pattern for the center inlet, and bottom pattern for the end inlet (Converse, 1974).

Limited monitoring of the field systems suggests that this method of effluent distribution retards clogging of the infiltrative surface. Several of the systems excavated did not show signs of a clogging mat developing, even after two years of operation. The application of effluent at a rate below the saturated conductivity of the soil in infrequent doses should allow rapid drainage and nearly continuous aerobic conditions at the infiltrative surface. Further work is needed to confirm this.

Restoring the Infiltrative Surface

Assuming that a system has been properly designed, sized, installed and maintained by necessary periodic tank pumping, the only reason it can fail is by excessive clogging or sealing of the soil in the absorption field. Short of installation of a new or extended seepage area, the only way to rehabilitate a failed system is to unclog the soil and restore its infiltrative capacity. Laboratory and field experience has shown that there are only two methods to do this. The two options are: 1) prolonged resting and 2) unclogging of the soil with chemical agents. Only these two approaches reach the problem where it is occurring, namely in the clogged portion of the soil.

Resting--

Several researchers have observed beneficial effects on infiltration following resting of clogged soil in laboratory columns (McGauhey and Winneberger, 1965; McGauhey and Krone, 1967; Jones and Taylor, 1965; Thomas, et al., 1966; De Vries, 1972; Perry and Harris, 1975; Harkin and Jawson, 1976). The practice of intermittent resting is recommended in HUD's (1974) handbook on "Methods of Preventing Failure of Septic Tank Percolation Systems." However, the increase in soil permeability produced by short-term resting is usually lost again within about 5 to 10 days after loading is resumed (McGauhey and Krone, 1967; Harkin and Jawson, 1976). The final infiltration rate is usually lower than that before resting is commenced (See Figures B-72 and B-73).

The main problem with resting is the provision of the long period necessary for beneficial resting, which may be days, weeks, or months, depending on the soil in question and the status of the clogging mat. If a new system is being installed or an older system is being upgraded, plans should be made to provide for future periods of resting by construction of a dual or alternating bed system. This obviously increases the cost of the basic septic system and is not always a viable alternative if the lot is too small to accomodate alternating beds. In some cases, the system may simply not drain rapidly enough. In finely textured soils, 45 to 60 cm (2.5 to 3.0 ft) of unsaturated soil below the infiltrative surface is necessary for the system to drain after effluent applications are stopped (Klein, et al., 1962; Bendixen, et al., 1962; Winneberger, et al., 1962). This depth of unsaturated soil is necessary to create a sufficiently high hydraulic gradient across the clogging mat. In areas of seasonally or permanently high water tables, drainage of the soil and reaeration may not occur at all during periods of resting.

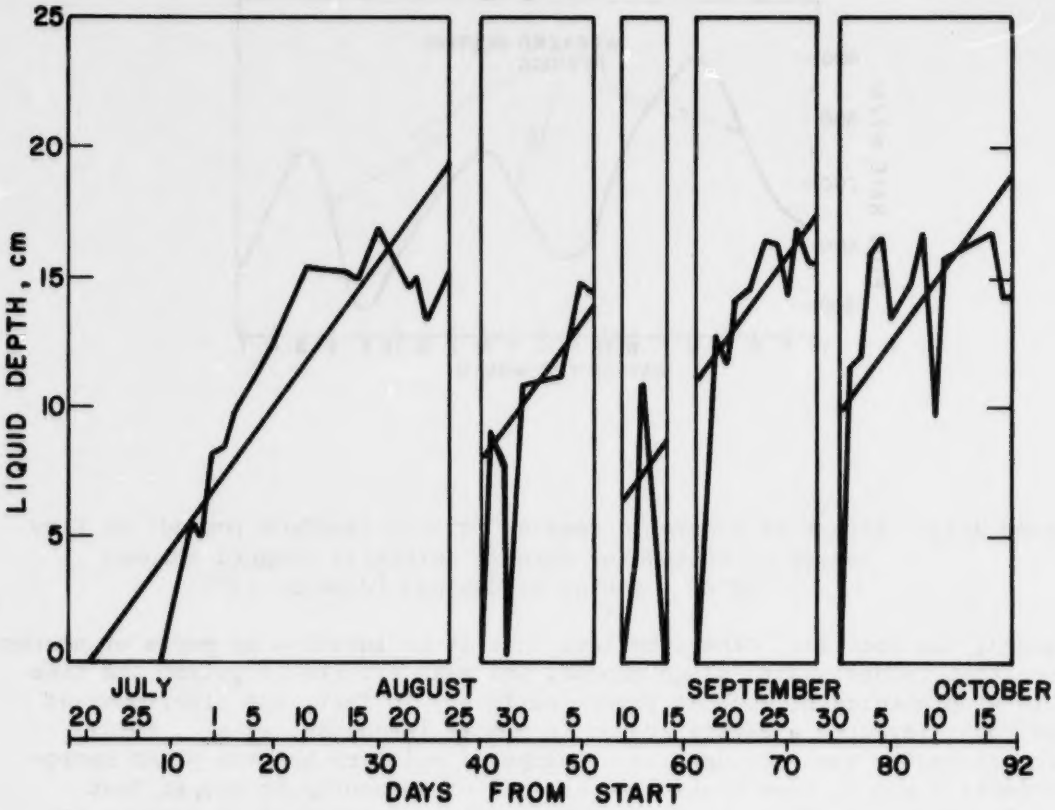

Figure B-72. Ponding of septic tank effluent in Hanford fine sandy loam with periodic resting (Winneberger, et al., 1962)

Chemical Treatment--

Even before the concept of restoration by resting was created, efforts had been made to restore the efficiency of ailing septic systems by additions of chemicals or other materials to the system through the household plumbing. Over the years a myriad of processes and agents have been used to try to improve septic tank performance.

The use of some techniques and materials is based on misconceptions of the problem and incorrect logic as to measures that could be taken to correct the situation. For example, many people, including officials administering health and plumbing codes and installers, believe that it is blockage of the perforated pipes or tiles in distribution lines by solids spilling over from badly maintained septic tanks that cause sluggishness or backups in septic systems. They think that snaking of the lines will restore functionality to the system. There is a simple test to determine whether blocked lines are the cause of a failure: if water is ponded in the seepage area, it has free access to the soil and the lines are not

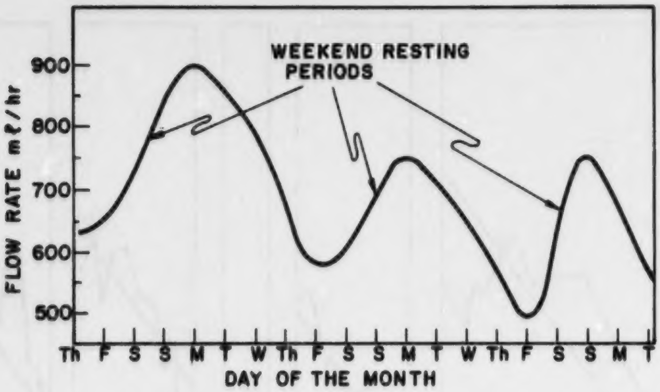

Figure B-73. Effect of anaerobic resting periods (surface ponded) on flow rates of wastewater through partially clogged columns (after 4 months of dosing) (Jawson, 1976)

plugged, the soil is. Others believe that it is invasion by roots of nearby trees that causes system sluggishness, and make efforts to poison the tree roots with chemicals, such as copper sulfate. In fact, the likelihood of tree roots invading a septic system is remote (Bendixen, et al., 1950). Although septic tank effluent is a source of moisture and the plant macronutrients N and P, tree roots need air and consequently do not, in fact tend to invade anaerobic ponded seepage areas. Inspection of systems installed so close to trees that the tree crowns spread over the drainfield area (so that an equivalent spread of the roots should be expected) have revealed that roots grow profusely in aerobic soils around, but outside, the clogged soil layers, but do not invade the ponded strata inside the clogged soil. Failure in such systems was again found to be due exclusively to soil clogging. The problem that has to be addressed, therefore, is soil clogging, although this may be complicated by other secondary factors.

Processes--Basically, there are three approaches to applying chemicals designed to unclog soils in drainfields: 1) through the household plumbing system to the septic tank and thence to the clogged field; 2) to the ponded water in the drainfield; and 3) directly to the clogged soil. Addition to the ponded water is less convenient than addition through the tank, but the principle is the same with less dilution of the chemical. Maximum effect of any additive can only be expected from direct addition to the clogged soil. What is more important is the avoidance of pernicious side effects of the chemicals on the operation of the tank, on the soil or on the groundwater quality.

Agents--The types of materials used to try to improve the performance of sluggish septic systems includes various categories of compounds: aggressive chemicals such as acids, bases, and strong oxidizing agents; biological agents or biochemicals such as yeasts, bacterial inocula, or enzymes; and surfactants. Proponents of these treatments usually recommend that the materials be added to the tank by flushing down drains or toilets. A few practitioners, realizing that the problem lay in clogging of the soil, have added enzymes, acids, or oxidizing agents directly to the drainfield or soil.

Effects--The Manual of Septic Tank Practice (1967) warns that over a thousand preparations which allegedly improve the performance of septic systems have been tested, but that not one has been found to be efficacious. Perhaps because no documentation of this contention is given, many manufacturers continue to concoct, market and advertise - often with the help of testimonials - a multitude of preparations that are purported to prevent or cure septic tank "sluggishness" or failure. Many are "guaranteed" to work. The producers of these nostrums usually suggest that a dirty tank is the problem and that addition of their product will clean the tank and create or stimulate bacterial action which will speed up liquefaction of the wastes in the tank, producing a more infiltrable, higher quality effluent. Such products are nationally advertised and sold. Most large department stores, hardware stores, and plumbing supply retail stores sell septic tank "cleaners" or "activators" of this type, despite the fact that the Manual states that "..there are no known chemicals, yeasts, bacteria, enzymes, or other substances capable of eliminating or reducing the solids and scum in a septic tank so that periodic cleaning is unnecessary. The addition of such products is not necessary for the proper functioning of a septic tank/soil absorption system."

To determine the efficacy of commercially available treatment, a series of 39 uniformly clogged sand columns were prepared (Jawson, 1976). The columns were continuously ponded for a total of 19 months before the treatments were tried. Flow rates were reduced to 0.02 cm/day.

The directions for use of most of the products recommend that a portion of the package contents be emptied into a drain or toilet and flushed. The amount added depends on the septic tank capacity. Based on the quantity stated for use, a proportional amount was added to 19 L (5 gal) carboys of septic tank effluent for each treatment. This "unclogging" potion was then added to the column in place of the raw septic tank effluent. In addition, some preparations were added directly to the clogged layer. After receiving direct applications, influents containing these preparations were used to load the columns.

The types of material tested included one or more examples of a "drain cleaner" containing strong sulfuric acid, a "septic tank and cesspool cleaner" containing a crude technical grade of sodium hydroxide, a "beneficial bacteria additive," a "root killer" ($CuSO_4 \cdot 5H_2O$ crystals), a "bacterial cleaner," a "bacteria enzyme" product, a "bacteria-enzyme activator," another "activator," a "bacterial sensation," a "septic tank conditioner," a complex commercial enzyme mixture designed for addition to drainfields, an emulsifying agent, a detergent, a chlorobenzene fat solvent, and

TABLE B-20. EFFECT OF VARIOUS "UNCLOGGING" TREATMENTS ON FLOW RATES THROUGH CLOGGED SAND COLUMNS (Jawson, 1976)

Column	Flow rate before treatment (ml/hr)	Subcrust moisture tension before treatment (cm)	Type of Treatment[1]	Flow rate 1 month after treatment (ml/hr)	Subcrust moisture tension 1 month after treatment (cm)
1	10	23.2	Clogging material removed from analysis	138	9
2	4	10.4	Bacteria (direct)	7	14
3	10	4.9	Enzyme (direct)	1.3	18.5
4	7	28.7	100 ml H_2O_2 (DuPont's compound 52A)	860	37.2
5	5	12.0	$CuSO_4$ "root killer" (direct)	6	10.0
6	30	19.0	Drain cleaner containing H_2SO_4 (direct) and sample taken for OM analysis	707	1.0
7	0.2	0	Aerated for 24 hours	10	21
8	0.1	0	None	12	32
9	1	18.7	Aerated for 96 hours	73	17.8
10	0.5	6.2	100 ml H_2O_2 (DuPont's Kastone) and sample taken for OM analysis	1530	9.1
11	8	14.7	Direct caustic followed by bacteria + enzyme	0	0
12	9	6.0	Bacteria (direct)	14	15.3

(continued)

TABLE B-20 (continued)

Column	Flow rate before treatment (ml/hr)	Subcrust moisture tension before treatment (cm)	Type of Treatment[1]	Flow rate 1 month after treatment (ml/hr)	Subcrust moisture tension 1 month after treatment (cm)
13	2	23.4	H_2O_2 (DuPont's stabilized)	1173	35.7
14	20	3.9	Bacteria (direct)	112	16.8
15	17	0	None	2.7	41.2
16	4	0	H_2O_2	584	0
17	5200	0	None (tap H_2O control)	5100	0
18	5000	0	None (tap H_2O control)	5300	0
19	2700	0	None (distilled H_2O control)	2090	0
20	2600	0	None (distilled H_2O control)	2080	0
21[2]	18	25	Unknown additive	686	16.2
22	25	40.1	Bacteria	12.5	39.3
23[2]	17	27.5	H_2O (DuPont's Kastone)	1557	31.6
24[2]	29	30.3	None	630	31.6
25[2]	24	27.8	Clogging material removed for analysis	950	32.4
26	17	16.8	Bacteria + enzyme	7.7	9.2

(continued)

TABLE B-20 (continued)

Column	Flow rate before treatment (ml/hr)	Subcrust moisture tension before treatment (cm)	Type of Treatment[1]	Flow rate 1 month after treatment (ml/hr)	Subcrust moisture tension 1 month after treatment (cm)
27	15	13.9	Effluent filtered to remove	6.3	15.3
28	10	40	Bacteria + enzyme	10.3	3.1
29	9	0	Bacteria	35.8	15.3
30	12	13.2	Bacteria	9.7	14.7
31	33	21.7	Enzyme	12.3	23.8
32	28	7.9	Bacteria	12.3	5.2
33	25	24.1	Direct caustic tank cleaner	21.4	28.9
34	33	19.4	H_2SO_4 drain cleaner	20.2	8.9
35	24	18.6	"Root killer" $CuSO_4$	13.5	15.8
36	4	15.7	None	5.8	17.0
37	9	20.1	Aerated for 72 hours and sample taken for OM analysis	238.6	0
38	28	23.6	Enzyme	10	4.6
39	18	11.5	Unknown	1	13.4

---

[1] Direct indicates that the product was added directly on the clogging mat in the same manner as H_2O_2.

[2] Columns 21, 23, 24 and 25 developed leaks that led to three 60-hour aeration periods.

various experimental grades of stabilized hydrogen peroxide.

When applied directly to clogged soils, only the sulfuric acid, the caustic alkali and the peroxides increased the soil permeability (See Table B-20). The peroxide preparations were invariably the most effective. Suitable applications restored the soils to almost the initial infiltration rates, and allowed increased percolation for several months after treatment. The acid and alkali also increased the infiltration rate initially, to an extent comparable with short-term resting, but not as effectively as the peroxides. However, the flow rate after one month had fallen to a lower level than that of the original clogged soil in the column treated with acid and to zero in that treated with alkali. The sodium hydroxide caused deflocculation and dispersion effects of the soil. Chemical analysis showed that the sulfuric acid and resting removed much less organic matter from the clogging layer than did the mild peroxide treatment.

None of the other materials produced any significant increase in infiltration. This is not surprising. The clogging mat is rich in heterotrophic bacteria and their enzymes so that addition of a few extraneous bacteria, bacterial spores, or enzymes cannot evoke a significant change. Even if the bacteria or enzymes present in the preparations happened to be adapted for optimum growth or action under the temperature, pH and redox conditions present in clogged soil, it is unlikely that they will be able to compete effectively with or supplement substantially the massive amounts of bacteria and enzymes already in the clogging zone. It is more likely that the commercial bacteria are not adapted to the peculiar environment of the clogged soil and the commercial enzymes are either inactivated by the sulfides present in the clogged soil or, as proteins, digested by the proteolytic bacteria in the clogging zone.

It is also not surprising that the surfactants or emulsifying agents do not increase infiltration. The rationale behind the use of surfactants is to reduce the surface tension in the ponded water and help it "slip through" the clogged soil or to help dissolve the hydrophobic fatty constituents of the clogging material. Some manufacturers apparently think that water-resistant fats and grease are a major constituent of the clogging material. Normally, this is not true. Chemical analyses show that lipids constitute only 10-15 percent of the organic material in clogged soils (See Table B-21). Moreover, there are already substantial levels of detergents present in undecomposed form in normal septic tank effluent. The low lipid content of the clogging zone also explains why the chlorobenzene does not unclog soil. There is also little sense in adding emulsifying agents or organic solvents to the system to dissolve fats and grease for the same reasons.

In practice, people use these preparations only after their system has developed problems, i.e., the soil is already clogged, and they normally add them to the tank through their household plumbing instead of directly to the clogged soil or the ponded water. Therefore, tests were conducted with clogged soil columns to see whether septic tank effluent treated according to manufacturers' prescriptions with various chemical and biochemical preparations would gradually unclog soil (Harkin and Iskandar, unpublished data, TABLE B-21. ORGANIC MATTER LEVELS IN THE CLOGGED ZONE AND ITS CHARACTERIZATION (Jawson, 1976)

Column	Treatment	Total organic matter	Fats, waxes & oils	Resins	Water soluble polysaccharides	Hemicellulose + cellulose	Protein	Lignin-humus	Acid soluble unknown
						%			
25	None (Subcrust aerated column)	3.5	15	5	12	0	3	12	53
15	None (Totally anaerobic column)	5.3	11	7	14	0	1	10	57
10	H_2O_2	0.41	17	8	21	0	1	8	35
6	H_2SO_4	1.5	35	11	10	0	6	19	19
27	Aerated for 72 hours	1.7	14	3	5	0	2	6	60
20	None (H_2O control column)	0.19	--	--	--	--	--	--	--

1977). In no case was an increase in infiltrative capacity obtained. On the contrary, sealing was accelerated and failure occurred more rapidly than with columns dosed with unamended effluent.

The only treatment that was found effective in unclogging soil was hydrogen peroxide when applied directly to the clogged soil. Most of the reagent is uselessly expended if applied to the ponded water or to the septic tank, because it is destroyed by reaction with materials in solution or suspension in the water. The reagent is much more effective if properly stabilized and correctly applied. It destroys most of the organic matter in the clogging zone, and it reestablishes aerobic conditions in the infiltration area, so that aerobic bacterial digestion of undissolved clogging material can take place. Peroxide treatments have also been found to be successful for rehabilitating failed absorption systems in the field (Harkin, et al., 1976; Harkin and Jawson, 1977). Peroxide treatments are the basis of the POROX ™ system for reviving failed septic systems (Harkin, 1977) which is now being commercially marketed.

Concern has been expressed that treatment with peroxide could cause organic matter, bacteria and other substances to move deeper into the soil and into groundwater with adverse affects, since a large amount of purification occurs as effluent passes through the clogged layer (McGauhey and Krone, 1967; Bouma, et al., 1972). To test this possibility, BOD and bacterial analyses were performed on column effluents before and after $H_2 O_2$ treatments (Jawson, 1976). Effluent samples were collected for several days following successful treatment to determine when and for how long these substances filter through sandy soil. Results from these columns are presented in Table B-22. The data do seem to indicate that both bacteria and organic matter are eluted in the treatment effluents. However, the concentrations of BOD and coliforms found do not appear to be high enough to cause alarm, especially since the columns were only 60 cm (2 ft) deep.

It appears that any strong oxidizing agent, acid or base could be a successful declogging agent. However, hydrogen peroxide appears to be the best choice based on very limited data thus far available. It works more effectively than other chemicals so far tried, with no major drawbacks apparent so far. One of its most positive properties is that it is reduced to water and oxygen, while most other chemicals tested have by-products which may be undesirable. Probably the biggest unknown is the secondary effects such as the release of toxic metals or virus during and immediately after peroxide treatment.

In all cases, direct application to the clogging mat was necessary for successful operation. This methodology treats the problem where it occurs, viz., in the seepage bed itself. Addition of a declogging agent to the septic tank itself would cause most or all of its beneficial effects to be dissipated through reagent degradation and/or dilution long before the problem symptoms are attacked.

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TABLE B-22. BOD AND TOTAL COLIFORM BACTERIA PRESENT IN COLUMN EFFLUENTS AFTER TREATMENT WITH HYDROGEN PEROXIDE (Jawson, 1976).

Column	BOD before treatment (mg/L)	Total coliform bacteria before treatment (#/100 ml)	Treatment	Days after treatment when sampled collected	BOD after treatment (mg/L)	Total coliform bacteria after treatment (#/100 ml)
Raw septic tank influent	110	380,000	None	1 to 3	82	250,000
6	25	265	H_2SO_4	1	6	90
6	25	265	H_2SO_4	2	40	2
6	25	265	H_2SO_4	3	20	0
23	12	14	H_2O_2	1	27	2
23	12	14	H_2O_2	2	45	770
23	12	14	H_2O_2	3	29	210

ALTERNATIVE SYSTEM DESIGNS FOR PROBLEM SOILS

The Mound System

There are many areas where the conventional septic tank-soil absorption field is not a suitable system of wastewater disposal. For example, sites with slowly permeable soils, excessively permeable soils, or soils over shallow bedrock or high groundwater do not provide the necessary absorption or purification of the septic tank effluent. However, these limitations often can be overcome by constructing the soil absorption field above the natural soil in a mound of medium sand fill (See Figure B-74).

There are several advantages to raising the soil absorption field. The fill below the absorption trenches within the mound provides additional soil material necessary to purify the wastewater before it reaches the groundwater at sites with shallow or excessively permeable soils. At sites with slowly permeable soils, the purified liquid is able to infiltrate the more permeable natural topsoil over a large area and safely move away laterally until absorbed by the less permeable subsoil. Also, the clogging mat that eventually develops at the bottom of the gravel trench within the mound will not clog the sandy fill to the degree it would in the natural soil. Finally, smearing and compaction of the wet subsoil is avoided since excavation in the natural soil is not necessary.

The mound system was originally developed in North Dakota where it became known as the NODAK disposal system (Witz, et al., 1974). The mounded design was proposed to overcome slowly permeable soil conditions by utilizing the more permeable topsoil. The seepage bed was constructed in gravel fill which was placed over the original soil after the topsoil had been removed. These systems were first installed in 1947 and have since appeared to function properly. However, monitoring data has been lacking, and design criteria based on the potential of the soil to absorb and conduct liquid have not been defined.

To develop sound design criteria, several mound systems of various designs were evaluated both under field and laboratory conditions (Bouma, et al., 1972; Bouma, et al., 1974c; Bouma, et al., 1975b; Magdoff, et al., 1974a; Magdoff, et al., 1974b). Six mound systems installed prior to 1970 were first investigated (Bouma, et al., 1972). These were similar in design to the NODAK system constructed to overcome both slowly permeable soil conditions and permeable soils over shallow creviced bedrock where groundwater contamination was a danger. The conclusions made from these initial investigations were that the gravelly fill was too coarse to provide adequate filtration where groundwater contamination was a problem and that inadequate distribution, sizing and/or construction was resulting in surface seepage during wet periods of the year.

To correct these shortcomings, changes were suggested in the design and construction of mounds. To provide better filtration and treatment, medium sand was specified for fill material. Loamy sands or sandy loams have better filtration properties but their potential for clogging is

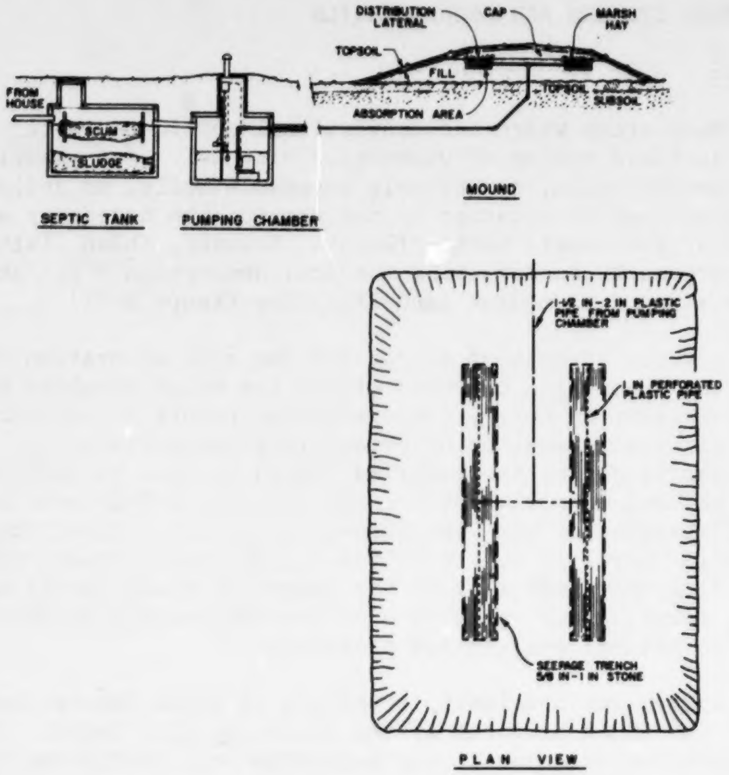
DISTRIBUTION LATERAL
CAP
MARSH HAY
TOPSOIL
FILL
TOPSOIL
SUBSOIL
ABSORPTION AREA
FROM HOUSE
SCUM
SLUDGE
MOUND
SEPTIC TANK
PUMPING CHAMBER
1-1/2 IN-2 IN PLASTIC PIPE FROM PUMPING CHAMBER
1 IN PERFORATED PLASTIC PIPE
SEEPAGE TRENCH 5/8 IN-1 IN STONE
PLAN VIEW

Figure B-74. A plan view and cross-section of a mound system for slowly permeable soils

greater (Bouma, et al., 1972). Crust tests were performed in the soils at each site to determine the soil's capability for conducting water, providing better sizing criteria. It was also specified that the original soil be disturbed as little as possible. The vegetation should be removed and raked, but the topsoil should remain in place and not be driven upon.

Full-scale mounds and laboratory models were constructed to test the modifications made. Models were designed to represent the vertical cross-section of a mound (Magdoff, et al., 1974a). Large 14.7 cm (6 in) diameter columns were filled with 15 cm (6 in) of gravel followed by 30 cm (12 in) of silt loam topsoil, 60 cm (24 in) of sand, 30 cm (12 in) of gravel and another 30 cm (12 in) of silt loam. This model represented a mound constructed over a shallow silt loam with underlying creviced bedrock (See Figure B-75). The upper gravel layer of each column was dosed with 8 cm/day (1.92 gpd/ft^2) of septic tank effluent. Monitoring included moisture tensions, gas sampling, redox potential measurements and liquid samples for COD, nutrients and bacteriological analyses with depth in the column.

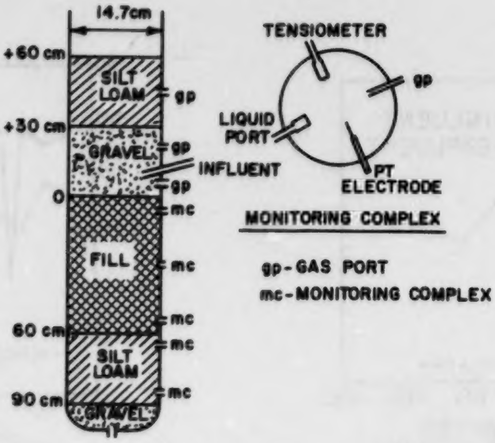

Figure B-75. Column model of mound over shallow creviced bedrock (Magdoff, et al., 1974).

The results of the laboratory studies indicated that a mound using 60 cm (24 in) of sand fill would provide adequate treatment (See Figures B-76, B-77, B-78 and Table B-23). Essentially complete removal of fecal indicators, and COD were realized with significant decreases in nitrogen and phosphorus. Only in situations where nitrate contamination of the groundwater is undesirable would such a design be technically unsuitable (Magdoff, et al., 1974b).

Several experimental mounds were constructed to test the suggested modifications. A rational design method was developed (Bouma, et al., 1975b). Different design considerations are dictated during the spring and fall in slowly permeable soils when natural water tables occur at a shallower depth than in summer and winter. Dimensions of the seepage trench are designed to avoid rising of the perched water table into the fill when the groundwater is high, and the total basal area of the mound should be sufficiently large to absorb and conduct the effluent downwards through slowly permeable subsoil horizons when the groundwater is low.

The calculation of the required basal area of a mound should be based on the K_{sat} of the least permeable soil horizon within 90 cm (3 ft) below the proposed site for the mound. Maximum flow is estimated by using 568 liters/day (150 gallons/day) per bedroom in the home served.

The second step in the design is to size the absorption system within the mound. The system should consist of one or a series of small parallel trenches, rather than one single absorption bed containing all the distribution laterals. Widely spaced trenches are superior than beds because they distribute the liquid over a larger area causing a smaller rise of perched

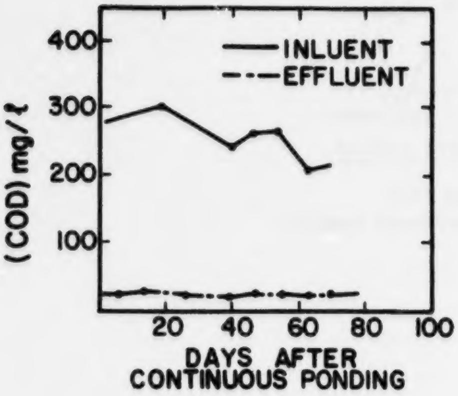

Figure B-76. Chemical oxygen demand (COD) of influent and effluent from column (Magdoff, et al., 1974b)

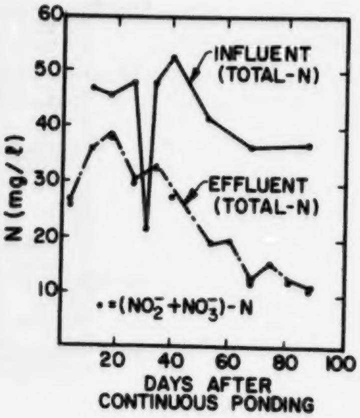

Figure B-77. N concentrations in influent and effluent from column (Magdoff, et al., 1974b)

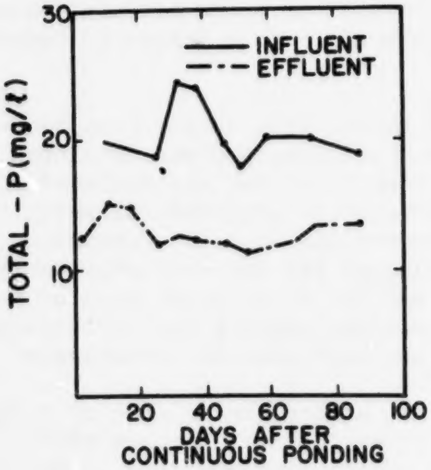

Figure B-78. Total-P concentrations in influent and effluent from column (Magdoff, et al., 1974b).

TABLE B-23. BACTERIAL COUNTS IN INFLUENT AND EFFLUENTS FROM COLUMNS (Magdoff, et al 1974b).

	Bacterial Group			
	FS[1]	FC[1]	TC[1]	TB[1]
	Counts/100 ml			
	Influent			
Range	$6.0 \times 10^{1} - 1.4 \times 10^{5}$	$3.0 \times 10^{4} - 6.5 \times 10^{5}$	$2.0 \times 10^{5} - 1.2 \times 10^{7}$	$4.3 \times 10^{8} - 9.7 \times 10^{9}$
Average	3.8×10^{4}	1.7×10^{5}	5.4×10^{6}	2.4×10^{9}
	Column Effluents			
Range	-	-	$<5 - 1.0 \times 10^{2}$	$2.0 \times 10^{6} - 2.7 \times 10^{8}$
Average	<5	<5	5	4.7×10^{7}

[1] FS - fecal streptococci; FC - fecal coliform; TC - total coliform, and TB - total bacteria.

liquid in the topsoil. A standard trench width (w) of 60 cm (2 ft) is recommended as a result of applying the Dupuit-Forchheimer assumption for horizontal flow applied to the topsoil (Childs, 1969). Figure B-79 shows a schematic cross section of one-half of a mound on top of a permeable topsoil of thickness h_o cm, resting on an "impermeable" subsoil B horizon. Infiltration of effluent from the seepage bed (I, cm/day) results in a rise of the groundwater in the topsoil. Below the center of the bed, the groundwater level is equal to the depth of the topsoil (h_o), below the edge of the bed the level is h_1. The calculation is designed to prevent groundwater from rising into the fill. Trench width (w) is calculated by the following equation, assuming I = 5 cm/day, h_o = 30 cm, and h_1 = 25 cm, and K from the K-curves for the particular soil.

$$(h_o^2 - h_1^2) = (I \ w^2)/2K$$

This analysis is approximate because: 1) the subsoil B is not impermeable; 2) the soil usually has some slope; and 3) the liquid is not applied as a steady flow, but as a dose. However, calculations do indicate the need for small seepage trenches rather than large seepage beds. The total bottom area of the trenches is calculated by using a liquid application rate of 5 cm/day (1.2 gpd/ft^2). The height of the gravel-filled trench should be at least 20 cm (8 in) above the natural soil surface to allow for sufficient liquid storage. If parallel trenches are used, their spacing is determined by the hydraulic characteristics of the underlying subsoil. The area between the trenches should be sufficient to absorb all the liquid contributed by the upslope trench, and trenches should be laid parallel to the slope. In more permeable soils, beds rather than trenches can be used because the water table is less likely to rise.

The selection of the most appropriate dosing regime is closely correlated with the type of fill, its hydraulic characteristics, and economic considerations. For example, dosing once every two days would require a

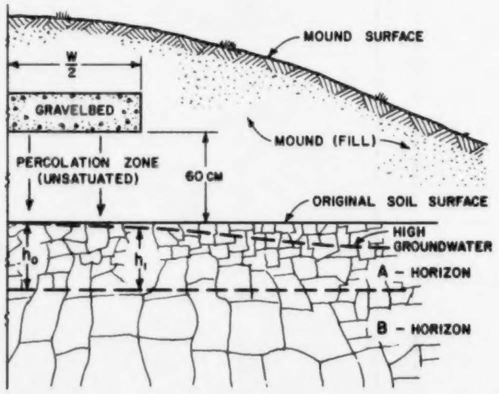

Figure B-79. Schematic cross-section through a mound system used to calculate the required width of the gravel bed in the mound.

relatively large pumping chamber, while instanteous introduction of this large quantity of liquid would probably result in saturated flow conditions in the fill and associated unsatisfactory purification (Bouma, et al., 1972). On the other hand, more frequent dosing, e.g. four times per day would result in better purification and require a smaller pumping chamber. However, the fill may then not have sufficient time to drain, and relatively high moisture contents at the interface of seepage bed and fill could lead to early clogging. Clogging can be at least partly removed by introducing a period of aeration in which clogging components are decomposed by aerobic bacteria. A long period between successive dosages would be advantageous in that it would provide relatively long intermittent periods of aeration.

The laboratory column experiments indicated that a once-a-day application of 8 cm of effluent was quite effective in removing fecal indicators, pathogenic viruses, BOD, and significant amounts of N and P (Magdoff, et al., 1974b; Green and Cliver, 1975).

Based on these column studies, the application rate of the field system to be used for sizing purposes was chosen to be 5 cm/day. This daily loading rate was derived from independent field measurements of soil moisture tensions below clogged seepage beds in sand (Bouma, et al., 1972). These tensions, which can be translated into flow rates by using the appropriate K-curves were approximately 25 mbar, corresponding with a flow rate of about 8 cm/day. The lower rate of 5 cm/day was used here to include a safety factor. The concept of using this rate is to ensure adequate infiltration of effluent, even if the seepage bed were to become clogged at some future time, assuming that the clogged layer would still allow percolation of 5 cm/day.

Five experimental mounds were constructed based on the rational design approach. Mounds I and II were constructed in September, 1971 in the slowly permeable Hibbing silt loam. Modifications based on experience with these systems were made in Mound III, also constructed in the Hibbing silt loam during June, 1972. Mounds IV and V were again modified to improve performance. Mound IV was constructed in the tight Almena silt loam, while Mound V was designed primarily to provide purification because of its location over a shallow, creviced bedrock. Salient characteristics of these systems appear in Table B-24.

As shown in Table B-24, the total bottom area of the mounds is much larger than the area required on the basis of the limiting conductivity of the subsoil. This discrepancy is caused by the requirement for 60 cm (24 in) of fill below the seepage trenches to purify the liquid. Addition of soil cover results in a mound that is 1.2 m to 1.5 m (4.5 to 5 ft) high at the center. Slopes at the side of the mound should not exceed 3:1 to insure stability which create large basal areas.

At slowly permeable soil sites, the depth of fill can be reduced because purification is not a problem. This was done in Mound IV (See Figure B-81). This reduces the basal area somewhat. The seepage areas within the mound are much smaller because they are based on flow characteristics of the sand fill. Mounds I and II were constructed as shown in

TABLE B-24. CHARACTERISTICS OF EXPERIMENTAL MOUNDS

		Home Size					Loading Rate		Basal Area		Seepage Area	
Mound	Construction Date	No. Persons	No. Bedroom	Soil Type	Percolation Rate cm/day (min/inch)	Minimum K_{sat} In Subsoil cm/day (gpd/ft^2)	Design[1] L/day (gpd)	Average[2] L/day (gpd)	Total m^2 (ft^2)	Minimum Required[c] m^2 (ft^2)	Total Area m^2 (ft^2)	Layout m (ft)
I	Sept. '71	3	3	Hibbing silt loam	4 (900)	1.5 (0.36)	1700 (450)	600 (150)	495 (5500)	39 (417)	36 (400)	single trench 1.2 x 30.5 (4 x 100)
II	Sept. '71	3	3	Hibbing silt loam	4 (900)	1.5 (0.36)	1700 (450)	560 (140)	495 (5500)	36 (390)	36 (400)	single trench 1.2 x 30.5 (4 x 100)
III	June '72	7	3	Hibbing silt loam	4 (900)	1.5 (0.36)	1700 (450)	860 (215)	495 (5500)	56 (597)	27 (300)	single trench 1.2 x 22.9 (4 x 75)
IV	Aug. '73	5	3	Almena silt loam	40 (90)	2.0 (0.5)	1700 (450)	880 (220)[d]	504 (5600)	41 (440)	22 (240)	three trenches 0.6 x 12.2 (2 x 40)
V	Aug. '72	2	3	Longrie sandy loam	120 (30)	70 (18)	1700 (450)	660 (174)	235 (2525)	13 (139)	28 (306)	bed 5.3 x 5.3 (17.5 x 17.5)

---

[1] Based on 568 L/day (150 gpd/bedroom)

[2] Measured rates from water meters

[3] Based on measured flow

[4] Homeowner damaged dosing tank allowing an unknown quantity of surface runoff water to be pumped into the mound during Spring, 1974.

Figure B-80, except the clay dam was not used and distribution was provided by the conventional 10 cm (4 in) perforated pipe. Some seepage occurred in localized spots at the sides of these mounds in the spring and fall, indicating that short circuiting was occurring either from unequal distribution of the effluent within the trench or inadequate absorption in the natural soil. Excavations into the side of Mound I after 2.5 years of operation indicated the occurrence of a slight compacted slimy surface layer formed from decomposed grasses at the original soil surface, even though grasses were cut and removed before applying the fill. The _in situ_ soil below the original soil surface was _not_ saturated and infiltration rates into the soil were, therefore, reduced (Bouma, _et al_., 1975b). The large diameter distribution pipe provided local overloading within the mound, and, coupled with the organic mat which developed at the original surface, seepage resulted.

To correct this problem, a clay dam was constructed inside Mound III to force the liquid over a wider area, and a pressure distribution network was tried to uniformly apply the liquid in the trench (See Figure B-80). This prevented any seepage, but monitoring standpipes extending down to the original soil surface showed 2.5 cm to 5 cm (1 to 2 in) of ponded water.

To further improve infiltration into the soil, the topsoil was first plowed under Mound IV. This was done when the soil was dry to a depth of 20 cm (8 in). The contour of the slope was followed, throwing the soil upslope to prevent an channeling. By using this construction technique, it was felt the clay barriers around the perimeter of the mound could be eliminated (See Figure B-81). This modification proved satisfactory (Bouma, _et al_., 1975b).

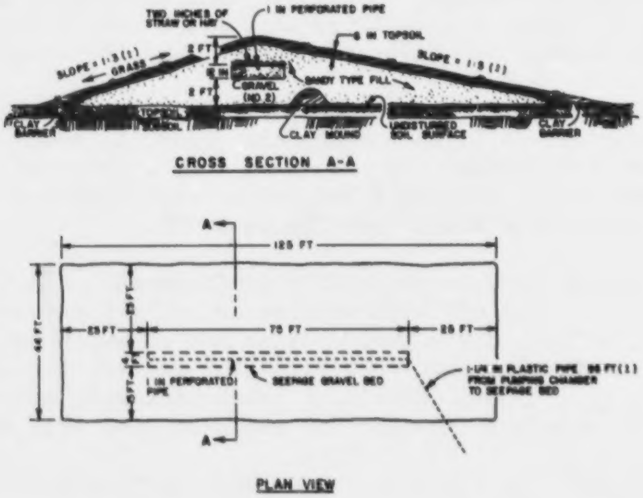
TWO INCHES OF STRAW OR HAY
1 IN PERFORATED PIPE
6 IN TOPSOIL
SLOPE = 1:3 (1)
GRASS
SLOPE = 1:3 (2)
2 FT
6 IN
2 FT
GRAVEL (NO 2)
SANDY TYPE FILL
TOPSOIL
SUBSOIL
CLAY BARRIER
CLAY MOUND
UNDISTURBED SOIL SURFACE
CLAY BARRIER
CROSS SECTION A-A
A
125 FT
44 FT
25 FT
75 FT
25 FT
1 IN PERFORATED PIPE
SEEPAGE GRAVEL BED
1-1/4 IN PLASTIC PIPE 98 FT (1) FROM PUMPING CHAMBER TO SEEPAGE BED
A
PLAN VIEW

Figure B-80. Section view and plan view of Mound III.

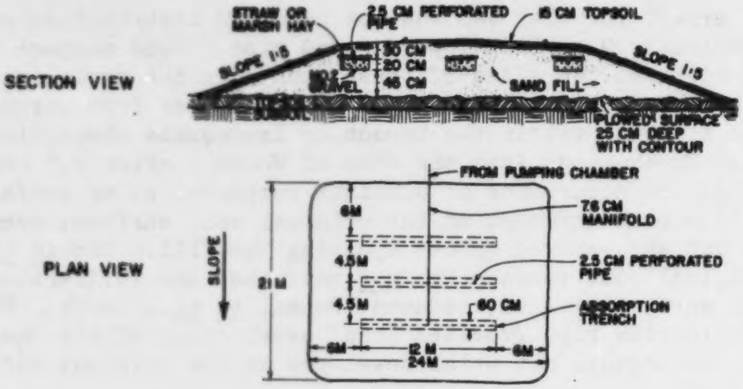
STRAW OR MARSH HAY
2.5 CM PERFORATED PIPE
15 CM TOPSOIL
SLOPE 1:5
SLOPE 1:5
30 CM
20 CM
45 CM
NO 2 GRAVEL
SAND FILL
SUBSOIL
PLOWED SURFACE 25 CM DEEP WITH CONTOUR
SECTION VIEW
FROM PUMPING CHAMBER
6M
7.6 CM MANIFOLD
4.5M
2.5 CM PERFORATED PIPE
4.5M
60 CM
ABSORPTION TRENCH
SLOPE
21 M
6M
12 M
6M
24M
PLAN VIEW

Figure B-81. Section view and plan view of Mound IV

A second modification in system IV was the construction of a more compact seepage system consisting of one manifold with three laterals, feeding three small trenches. The trenches were built perpendicular to the direction of the slope. The distance between the trenches was 4.5 m (15 ft), which was considered adequate to allow infiltration of applied liquid during downslope movement before the soil area beneath the second trench was reached. One advantage of this system was its more attractive shape (21 m by 24 m) as compared with Mounds I through III (13 m by 38 m), even though total bottom areas are identical. Seepage has not occurred in Mound IV, even though several problems were encountered in operation due to a careless homeowner who damaged the septic tank and dosing chamber while landscaping the yard. Unknown quantities of surface runoff water have seeped into the dosing chamber and have been pumped into the mound.

Mound V was constructed prior to the inception of plowing, but because of the more permeable soil, no seepage has occurred, even through the clay barriers around the perimeter of the mound were eliminated. In addition, the more permeable soil allowed a bed rather than trenches to be constructed, making a more attractive mound (See Figure B-82).

All five mounds were monitored for performance. Liquid samples were taken of the septic tank effluent applied and of any seepage that occurred (Bouma, _et_ _al_.,1975b). Since purification was of major concern with Mound V, soil samples beneath the mound were taken for bacteriological analyses as well (Bouma, _et_ _al_., 1974c). Results of these analyses appear in Table B-25 showing that adequate purification was achieved with this design.

Winter operation is a concern with these systems because of the danger of freezing. Thermocouples were installed at various points within Mounds

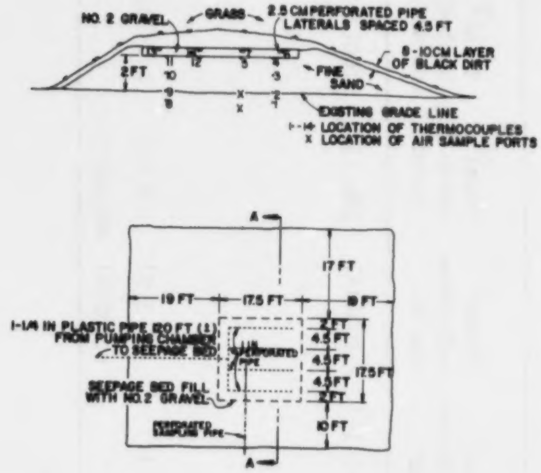
NO. 2 GRAVEL
GRASS
2.5 CM PERFORATED PIPE LATERALS SPACED 4.5 FT
8-10CM LAYER OF BLACK DIRT
FINE SAND
2 FT
11
12
5
6
10
3
9
8
X
2
1
EXISTING GRADE LINE
1-14 LOCATION OF THERMOCOUPLES
X LOCATION OF AIR SAMPLE PORTS
A
17 FT
19 FT
17.5 FT
19 FT
1-1/4 IN PLASTIC PIPE 120 FT (1) FROM PUMPING CHAMBER TO SEEPAGE BED
PERFORATED PIPE
2 FT
4.5 FT
4.5 FT
4.5 FT
2 FT
17.5 FT
SEEPAGE BED FILL WITH NO. 2 GRAVEL
PERFORATED SAMPLING PIPE
10 FT
A

Figure B-82. Section view and plan view of Mound V showing location of thermocouples and gas sampling ports (Bouma, et al., 1974c).

III and V as shown in Figures B-80 and B-82. Results from this monitoring indicated that although freezing temperatures did occasionally occur, freezing did not result because the trench and distribution pipe drained between dosings (See Tables B-26 and B-27. However, a tighter fill material (such as silt loam) over the trenches is recommended to improve insulation.

Based on the success of the experimental mounds, their use for on-site disposal was recommended for sites with: 1) slowly permeable soils; 2) permeable soils over shallow creviced or porous bedrock; and 3) permeable soils with high water tables. A detailed manual describing the design and construction procedures was written for use by regulatory personnel and contractors (Converse, et al., 1975b, 1975c, 1975d

Curtain and Underdrain Systems

Hydrodynamic Design Considerations--

The placement of drains and trenches in small scale waste disposal systems are geometrically and hydraulically similar to the system shown in Figure B-83. (Only half of the subsurface system is shown due to symmetry about the z-axis). Two design questions are considered, with due consideration given to the satisfactory disposal of the liquid waste.

In order to ensure adequate purification of the effluent (Bouma, et al., 1972), effluent draining from the trench must pass through a partially saturated zone of sufficient thickness to allow for treatment of the wastewater. Current health codes require a minimum distance of 0.9 to 1.2 m

TABLE B-25. MONITORING DATA FOR MOUND V, DERIVED FROM LIQUID SAMPLES TAKEN AT DIFFERENT LOCATIONS IN THE MOUND (Bouma, et al., 1974c).

Liquid samples	NH_4-N (mgN/L)	NO_3-N + NO_2-N (mgN/L)	ORG N (mgN/L)	Total P (mgP/L)	BOD (mg/L)	COD (mg/L)	Total[1] coliforms (no/ml)	Fecal[1] coliforms (no/ml)	Fecal[1] strep (no/ml)	Total[2] bacteria (no/ml)X10^5
Septic tank effluent	56±9	<1	6±2	15	90±35	256±80	37,000(15)	2,500(14)	100(13)	100(13)
Fill-soil interface[2]	<1	66	<1	14	-	-				
12 cm into topsoil[2]	<1	62	2	13	-	-				
20 cm into topsoil[2]	<1	54	5	8	-	-				
Collection pipe	2±1	54±6[3]	<1[3]	8	0	42	54(13)	5(7)	1.8 (9)	2.2(11)
								<0.02(4)	<0.02(3)	

[1] Medium values were obtained from log-probability graphs. Numbers in parentheses indicate the number of samples.

[2] Sampling 3/8/73 of porous cups inserted in the soil (unsuitable for bacterial analyses due to filtering action).

[3] Significant differences were observed between measurements in December (30 ppm NO_3-N; 8 ppm NH_4^+; 10 ppm Org N) and May (57 ppm NO_4-N; 3 ppm NH_4-N; 0.3 Org N) indicating a relatively low biological activity in the winter period.

TABLE B-26. TEMPERATURES MEASURED IN MOUND III FOR SEPT. 1972 TO MAY 1974 (Bouma, *et al*., 1975b)

Positions[1]	Temperature, °C											
	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1	2	1	1	2	5	12	13	15	13	10	6	3
2	1	1	1	1	6	13	14	16	13	9	6	2
3	-1	0	0	2	7	15	17	17	13	8	4	1
4	-1	-1	0	2	7	15	16	17	12	8	4	1
5	-2	-2	-1	2	7	17	18	18	12	7	2	-1
Average ambient	-10	-9	-2	4	9	19	20	20	14	7	0	-9

[1] Positions are located on cross-section of Mound III in Figure B-82.

TABLE B-27. TEMPERATURES OF MOUND V (Bouma, et al., 1974c).

	Temperature (^{o}C)			
Position	Nov. 12, 1972	Dec. 14, 1972	Jan. 29, 1973	May 18, 1973
1	11	8	5	10
2	11	7	5	10
3	10	7	4	10
4	10	6	4	11
5	11	7	5	11
6	8	2	2	10
7	8	4	3	10
8	11	8	5	9
9	12	7	4	10
10	11	7	4	10
11	10	6	4	11
12	11	6	5	11
13	9	3	2	10
14	9	4	3	11

(3 to 4 ft) between the source of the effluent, i.e., the trench, and the phreatic surface of the fully saturated groundwater below. However, in order to maintain generality, the design procedure will include an unspecified unsaturated thickness - T.

Design problem no. 1--Consider a waste disposal trench (See Figure B-83) of given width, 2w, located at a given distance, d, above a horizontal, impermeable layer. Liquid waste is released steadily from a bottom of this trench at a given infiltration rate, p_o (flow rate/unit area of the trench bottom), into a soil with a given hydraulic conductivity, K. Upon reaching the saturated zone the effluent flows toward the two drains, in which the flow depth, a, (which may be negligible) is maintained.

Find the drain spacing, 2b, which will provide a sufficient thickness, T, of the unsaturated zone to meet health standards.

Design problem no. 2--consider a waste disposal trench (See Figure B-83) of given width, 2w. Liquid waste is released steadily from the bottom of this trench at a given infiltration rate, p (flow rate/unit area of the trench bottom), into a soil with a given hydraulic conductivity, K. Upon reaching the saturated zone the effluent flows toward the two drains which are spaced a given distance, 2b, apart, and in which a given flow depth, a, (which may be negligible) is maintained.

Find the distance, d, which must separate the trench and the impervious layer to meet health standards.

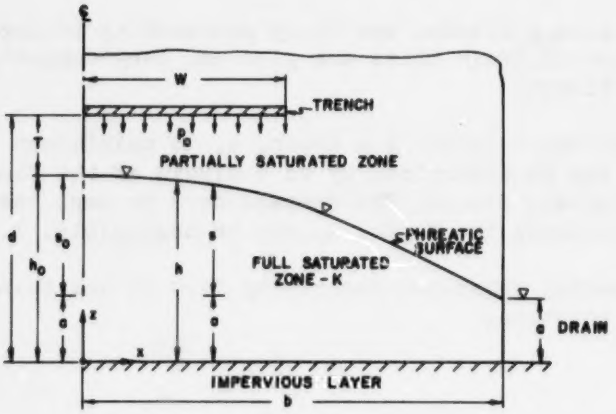

Figure B-83. Subsurface waste disposal system.

In order to solve these design problems it was necessary to devise an equation that described the shape of the groundwater mound which is formed below the trench. Once the shape of the phreatic surface is known, the elevation of the peak of the mound, h_0, can be calculated such that the unsaturated zone, T, will be of sufficient thickness to satisfy health standards.

Development of the Equation Describing the Phreatic Surface--

In order to develop the equation describing the shape of the phreatic surface, the non-linear Dupuit-Forchheimer approximation was employed. Details of this method of analysis may be found in the literature (Decoster, 1976). Only a summary will be offered here.

The following simplifying assumptions were made to establish a basis for the analysis.

1. The porous medium above the impervious layer is homogeneous and isotropic.

2. The impervious stratum is horizontal.

3. The effluent is released uniformly across the entire width of the trench at a constant rate. It percolates uniformly and vertically downward through the unsaturated zone.

4. The porous skeleton is considered rigid, and the water is essentially incompressible.

5. The flow obeys Darcy's Law.

6. The drainage ditches are fully penetrating to the impervious layer or if drain tiles are planned, they completely intercept the effluent.

7. A fixed water level at a depth, a, is maintained in the drains, which may be determined by an analysis of the flow conditions in the drainage ditch. The seepage face is neglected. Under some circumstances the depth, a, may be negligible, i.e., $a \simeq 0$.

The fundamental equations describing flow at any point in the field are the continuity equation,

$$\nabla \cdot \vec{q} = 0 \qquad (1)$$

and Darcy's Law,

$$\vec{q} = -K\nabla\phi \qquad (2)$$

where $\vec{q}$ is the vector specific discharge, K is the hydraulic conductivity, and ϕ is the piezometric head. Therefore,

$$\phi = \frac{p}{\gamma} + z \qquad (3)$$

where p is the fluid pressure, γ is the specific weight of the effluent and z is the vertical coordinate, measured upward from the impervious base. By substituting Eq. (2) into Eq. (1) and integrating the resulting Laplace's Equation over the depth, z, the Dupuit-Forchheimer Equation may be obtained. The sole additional restriction in the analysis is that the pressure distribution throughout the saturated zone must be essentially hydrostatic. In other words, the flow must be essentially horizontal and the phreatic surface may slope only moderately in the flow direction. The resulting equation for steady flow with infiltration, p, as developed by Decoster (1976) is:

$$\frac{d^2h^2/2}{dx^2} = -\frac{p}{K} \qquad (4)$$

with boundary condions

$$\begin{aligned} &\frac{dh(0)}{dx} = 0 \\ &h(b) = a \\ &p = p_o \qquad 0 \le x \le w \\ &p = 0 \qquad w < x \le b \end{aligned} \qquad (5)$$

and compatibility conditions at x = w to assure that there is no discontinuity in either the phreatic surface elevation or the flow rate at this section.

Solutions to Eq. (4), subject to Eq. (5) and the compatibility conditions are:

$$\frac{h}{w} = \left\{\frac{p_o}{K}\left(\frac{2b}{w} - \left(\frac{x}{w}\right)^2 - 1\right) + \left(\frac{a}{w}\right)^2\right\}^{1/2} \quad \text{for } 0 \leq x \leq w \qquad (6)$$

and

$$\frac{h}{w} \quad \left\{\frac{p_o}{K}\left(\frac{b}{w} - \frac{x}{w}\right) + \left(\frac{a}{w}\right)^2\right\}^{1/2} \quad \text{for } w < x \leq b \qquad (7)$$

These two equations decribe the elevation of the phreatic surface, h, as a function of position, x, with the constant values, p_o, K, b, w and a describing the effluent rate, hydraulic conductivity of the porous medium, and the geometry of the system components.

In as much as we are primarily concerned with the elevation of the phreatic surface at x = 0, i.e., at the peak of the mound, Eq. (6) can be evaluated at this point to give:

$$\frac{h_o}{w} = \left\{\frac{p_o}{K}\left(\frac{2b}{w} - 1\right) + \left(\frac{a}{w}\right)^2\right\}^{1/2} \qquad (8)$$

Eq. 8 is the basis for the design of the system since it contains the terms that describe the flow of the effluent, p_o, the hydraulic conductivity of the porous medium, K, and the terms that describe the system geometry, h_o, w, b and a. In fact, one could rely entirely on this equation to determine either the unknown distance, d, between the trench and the impervious bottom (since d = h_o + T) as explained in Design Problem 1, or the drain spacing, 2b, as described in Design Problem 2.

However, to simplify the process of solving Eq. (8), a graphical presentation is preferable, and this can best be accomplished by dividing both sides of Eq. (8) by $(p_o/K)^{1/2}$, and introducing $s_o = h_o - a$. This results in:

$$\left(\frac{s_o}{w} + \frac{a}{w}\right)^2 \frac{K}{p_o} = \left(\frac{2b}{w} - 1\right) + \left(\frac{a}{w}\right)^2 \frac{K}{p_o} \qquad (9)$$

or

$$(S + A)^2 = \frac{2}{B} - 1 + A^2 \qquad (10)$$

where the three dimensionless parameters are identified as follows:

$$S = \frac{s_o}{w} \frac{K}{p_o}$$

$$A = \frac{a}{w} \frac{K}{p_o} \qquad (11)$$

$$B = \frac{w}{b}$$

Eq. (10) which consists of only three variables is presented in Figure B-84.

Decoster (1976) also developed solutions to several unsteady flow problems and sought a verification of the steady flow case considered here. By comparing the Dupuit-Forchheimer results with those from a finite-element analysis and a Hele-Shaw analog, he concluded that the Dupuit-Forchheimer approach is very satisfactory for $A \geq 4.0$ while for values of $A \leq 4.0$ the phereatic surface elevation may be underestimated by as much as 15 percent.

Design Procedure--

With Figure B-84 and Eq. (11), it is now possible to solve the two design problems previously posed.

Design problem 1--Let us assume that a given disposal system consists of a 250 cm wide trench placed 200 cm above the impervious stratum, with the water elevation in the drains not exceeding 50 cm. The rate of effluent discharge is limited to 1 cm per day (0.2 gpd/ft^2) and the hydraulic conductivity is everywhere equal to 3 cm per day.

The problem is to find the appropriate drain spacing, 2b. In this example, calculation of S and A, based on the given data, is as follows:

$$S = \frac{s_o}{w} \frac{K}{p_o} = \frac{(200-90-50)}{125} \frac{3}{1} = 0.82$$

$$A = \frac{a}{w} \frac{K}{p_o} = \frac{50}{125} \frac{3}{1} = 0.70$$

Therefore from Figure B-84, B = 0.85 and 2b = 295 cm.

The drains may be spaced less than 295 cm apart, and a satisfactory distance T will be maintained between the trench and the phreatic surface. In fact, since A is less than 4.0, it might be appropriate to design for a somewhat smaller value of b. However, it would appear that sufficient spacing should be maintained to assure that none of the effluent from the trench is released beyond either of the drains, i.e., outside of the drainage system.

Design problem 2--Assuming that a second disposal system consists of a 220 cm wide trench with water elevations in the drains so small as to be negligible, e.g., 5 cm. The drains are spaced 440 cm apart. The rate of effluent discharge is limited to 0.8 cm per day (0.16 gpd/ft^2), and the hydraulic conductivity is everywhere equal to 3.2 cm per day. The problem is to find the elevation of the bottom of the trench above the impervious layer required to assure that health standards are met.

In this example calculations of A and B, based on the given data, are as follows:

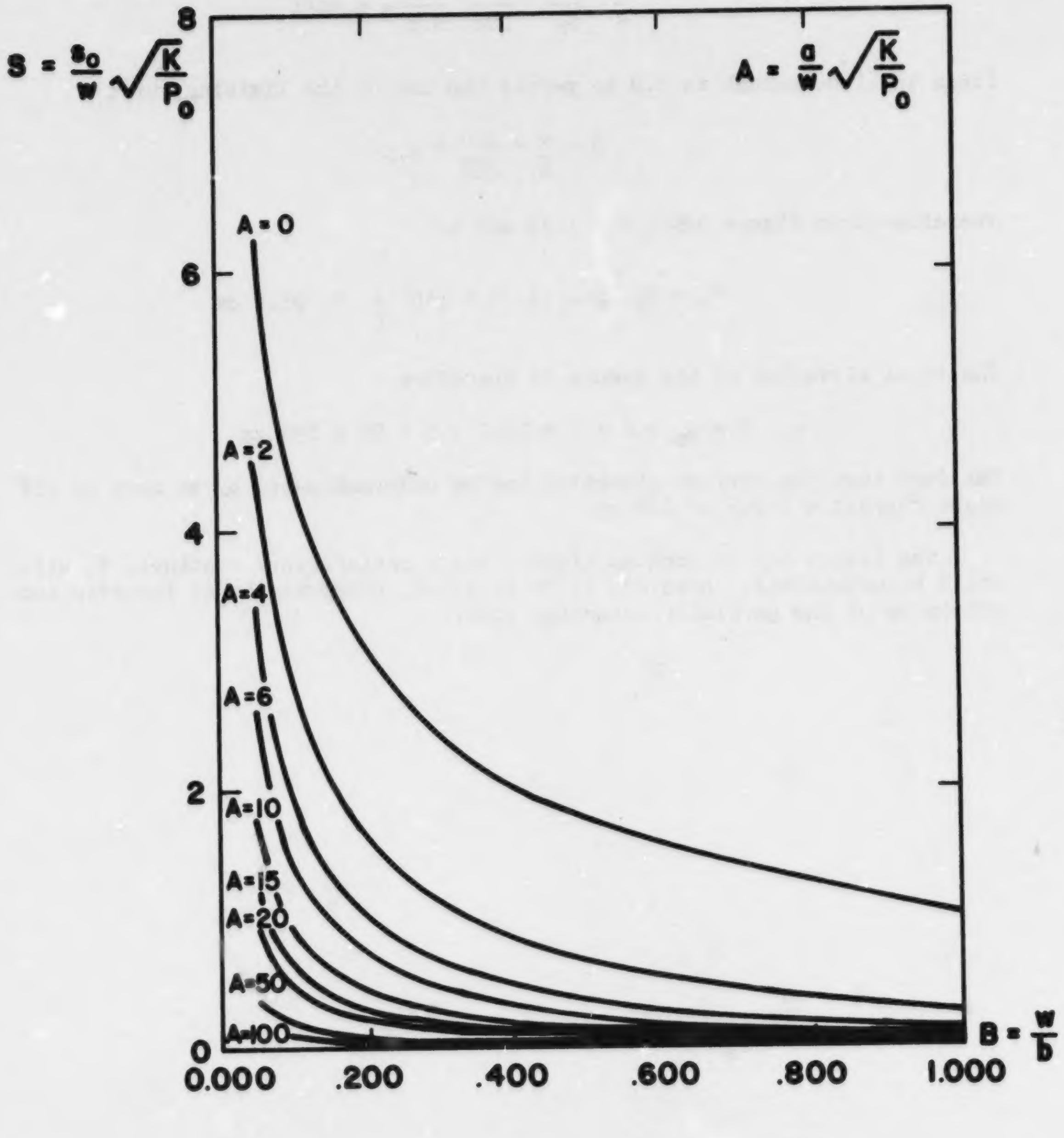

Figure B-84. Subsurface waste disposal design graph (Decoster, 1976).

$$A = \frac{a}{w} \quad \frac{K}{p_o} = \frac{5}{220} \quad \frac{3.2}{0.8} = 0.0454,$$

(this is close enough to 0.0 to permit the use of the limiting curve)

$$B = \frac{w}{b} = \frac{110}{220} = 0.5$$

Therefore from Figure B-84 , S = 1.73 and so

$$s_o = S_w \frac{p_o}{K} = 1.73 \cdot 110 \quad \frac{1}{4} = 95.2 \text{ cm}$$

The total elevation of the trench is therefore

$$d = s_o + a + T = 95.2 + 5 + 90 = 190 \text{ cm}$$

The fact that the surface elevation may be underestimated by as much as 15% might suggest a depth of 220 cm.

The trench may be located higher, and a satisfactory distance, T, will still be maintained. However, if it is lower, there may be an insufficient thickness of the partially saturated zone.

APPENDIX C

THE FATE OF BACTERIA, VIRUSES AND NUTRIENTS IN SOIL

Since the link between disease outbreaks and the presence of pathogenic organisms in waste-contaminated drinking waters was first established, there has been considerable effort to reduce the numbers of bacteria and viruses in potable waters by proper treatment of wastewaters. In addition, eutrophication from excessive nutrient concentrations in surface waters and the public health hazard of increased nitrate concentrations in groundwater has directed much attention to methods of controlling point and non-point discharges of nitrogen and phosphorus.

From the standpoint of public health, removal of bacteria and viruses is the most critical function of a wastewater treatment system. Yet, the role of bacteria in the biodegradation of the wastes also must be recognized. Without bacterial enzymes, the hydrolysis of organic solids and their subsequent oxidation would not occur. The biodegradation is the very basis of biological waste treatment, and so bacterial growth and metabolism are to be encouraged under controlled conditions. It is only in the final treated wastewater that there is a concern for eliminating bacteria and viruses which pose a potential danger to public health. This becomes a particular concern when partially treated wastes are applied to the soil for final disposal. Therefore, knowledge of the fate of bacteria and viruses in soil and the mechanisms of removal, is essential for preventing contamination of groundwater in areas employing subsurface wastewater disposal systems.

Nutrients in wastewater are also of concern. Nitrogen, in the form of nitrate or nitrite, found in private water supplies has been linked to cases of methemoglobinemia in infants. Also, accelerated eutrophication can occur if nitrogen and phosphorus are allowed to reach surface waters.

Therefore, the fate of bacteria, viruses and nutrients in the soil is of paramount importance. A knowledge of the soil capabilities to remove these contaminants, as well as the mechanisms involved, is necessary to properly design and operate soil absorption systems for wastewater disposal.

THE FATE OF BACTERIA IN SOIL

Wastewater Bacteria and Their Detection

Human Intestinal Bacteria--

The normal intestinal tract of man and all other warm-blooded animals contains a characteristic population of bacteria. At birth the tract is sterile, but it is quickly invaded (certainly within the first day of life) by bacteria from the mother's skin, food and water, and other objects put into the infant's mouth. With the first food in the tract, these bacteria grow and the typical numbers and great variety of bacteria in the intestinal flora are quickly established. All of this is quite normal, unless there is a gross unsanitary handling of the infant, involving the risk of intestinal diseases. In fact, it serves to establish the natural intestinal flora that will persist throughout the child's life, which is believed both beneficial for the mechanical functioning of the gut and in vitamin synthesis.

While the infant is fed milk, the dominant bacteria are lactose fermenters, e.g., fecal coliforms and lactic acid bacteria, including the fecal streptococci. Later, as non-sterile foods in great variety are ingested, many other types of bacteria are introduced, but they do not displace the fecal streptococci and fecal coliforms. These bacteria co-exist in a complex mixture of many types of bacteria, somewhat affected in numbers, but not in kinds, as diet varies throughout life (Haenel, 1961).

Many of the bacteria of this mixture are the same kinds as those found in natural soils and waters. These intestinal bacteria become the general flora in sewage and do not pose any danger to either health or the environment. In bacteriological testing of sewage, they appear in the Total Bacteria Count (Standard Methods, 1971). It is recognized that the count is far from "total," but it is representative of the main bacterial populations involved in general biodegradation.

Indicator Bacteria--

Superimposed on these normal groups of intestinal bacteria may be pathogens, if the individual is infected and shedding them via the feces. Detection of these pathogens is difficult, due to their variety and low populations. Therefore, to protect the public health, it is common practice to determine if fecal contamination has occurred, thus indicating a potential presence of pathogenic bacteria.

Fecal coliforms and fecal streptococci are commonly used as indicators of fecal pollution, since they satisfy four important criteria:

1. They are always present in very high numbers in the feces of man and other warm-blooded animals and do not naturally occur in high numbers elsewhere in nature.

2. They are able to survive for sufficiently long periods of time outside the body to be still present in wastewaters under treatment.

3. They can be monitored because quantitative methods for their detection and quantitative pollution standards based on these methods have been developed.

4. They generally outnumber the pathogens, and thus detection is more likely to be successful. They can serve as "indicator bacteria: because where they are found there is a possible presence of pathogens.

However, there are some limitations to the interpretation of indicator bacteria counts as an indicator of significant pollution. First, in natural soils and waters, there may be contamination from wild animals, resulting in a low background count (Geldreich, et al., 1962). In an effort to distinguish between animal and human sources of the indicator bacteria, Geldreich (Geldreich, et al., 1964, Geldreich, 1967) proposed a ratio of fecal coliform to fecal streptococci of 4:1 or greater as indicating human intestinal pollution, whereas a 0.6:1 ratio implicates livestock, poultry, cats, dogs and rodents. But this distinction has never been widely tested and has not been incorporated into Standard Methods (1971). Another approach for determining the identity of human vs. animal intestinal pollution is based upon determination of the species of fecal streptococci in the sample. *Streptococcus faecalis* occurs in both animals and humans, but predominates in humans, whereas it is found in relatively low numbers in lower animals. Therefore, only higher and repetitive counts should be considered to indicate human source pollution. This consideration is useful when following decreasing numbers of indicator bacteria from the high counts at the waste source through stages of treatment and final disposal.

There is one additional point to be kept in mind in the interpretation of coliform bacteria counts, that is the distinction between fecal coliform (FC) and total coliform (TC). Standard Methods (1971) defines the coliform group of comprising "...all of the aerobic and facultative anaerobic, gram negative, non-spore-forming, rod-shaped bacteria which ferment lactose with acid and gas formation within 48 hr. at 35°C." This total coliform group is heterogeneous and includes bacteria of several genera: *Escherichia, Enterobacter, Citrobacter, Klebsiella, Hafnia, Pectobacterium*, and *Erwinia*. Although any of these coliforms may occur in the human intestinal tract and, therefore, in the feces, some of them also occur in soil and water and on green plants. Their best growth temperature is lower than that of *Escherichia coli,* i.e., about 37°C. The other coliforms prefer 30 to 35° C, which is, of course, still well within the optimum growth range for *E. coli.* Thus, the total coliform count run at 35°C, includes both *E. coli* and other coliforms. However, the higher more optimum temperature for *E. coli* allows one to make a separate count by running tests at 44.5°C in a medium favoring *E. coli.* This represents the fecal coliform (FC) count which generally account for only about one fifth of the total coliform (TC) count on fresh feces or raw sewage. The significance of the total coliform count varies. When applied to natural soils or green plant samples, the TC count reflects mainly the *Enterobacter, Pectobacterium,* and *Erwinia* members of the complex, but when applied to human feces, the count has the same significance as the fecal coliform count. When applied to unpolluted soil and water, it reflects the low background count attributable to a scattering of wild animals and possibly some surviving or free-living coliforms. Thus, there is some significance in high total coliform counts. For the final effluent or the soil absorption bed samples, where counts are low, greater reliance is given to the FC count. Even so, testing for both *E. coli* and *S. faecalis* (fecal streptococci, FS) is even more reliable and has been used in most of these investigations.

When the indicator bacteria are released into soil at the disposal site, a new complication arises. They may survive temporarily, or grow if conditions permit, or die off in competition with the soil bacteria. Rudolfs, et al., (1950) have reviewed these aspects of sewage bacteria in soils. More recently, others (Geldreich, 1967; Van Donsel, et al., 1967; and Gerba, et al., 1975) have discussed conditions which may determine survival or die off of the indicator bacteria in soils. Many of these conditions are operative in the sewage/soil systems reported in this investigation and will be discussed later.

Pathogens--

Man is subject to several intestinal or enteric diseases, meaning that the gut is the infected site. Typhoid and paratyphoid, food poisoning and other diarrhoeas and dysenteries, caused by several bacterial, viral and amoebic pathogens, are examples of enteric diseases. The causal agents, whether animal parasites, pathogenic bacteria or viruses, may be present at times in feces and in wastewaters (Craun, 1975).

There are three groups of pathogenic bacteria which can be monitored directly in sewage studies; i.e., *Salmonella* spp., *Pseudomonas aeruginosa* group and *Staphylococcus aureus* group. Their incidence in healthy humans are: salmonellae 0.04% (Geldreich, 1970; Hall and Hauser, 1966; Craun, 1975); *P. aeruginosa* 12-14% (Hoadley and McCoy, 1968); and *S. aureus* about 18% in adults and 40+% in children under 2 years old (Elek, 1959; Ziebell, et al., 1975b; and Smith and Crabb, 1975). Because of their incidence and significance to public health, these were the pathogens chosen for monitoring in this investigation. The rationale of choice was:

1. Significance of salmonellae: If related to even a small number of disease cases, its proven occurrence constitutes an outbreak and is reportable to authorities;

2. Usefulness of *P. aeruginosa* and *S. aureus* monitoring: When present in feces in sufficient numbers to allow detection, reduction in numbers during waste treatment and disposal is easily monitored.

3. Quantitative measurement: Physiology of any of the above species allows selective media and culture methods on which to base detection and enumeration in the presence of other sewage and soil bacteria; and

4. Low human risk: The risk of human infection during laboratory handling is present but low, and safeguards can be provided by employing proper techniques.

Salmonella spp.--The salmonellae comprise a large and complex group of species and/or varieties, whose classification is based upon serological typing (Bergey's Manual, 1974). A general term for salmonella infection is salmonellosis. Some salmonellae are linked to specific diseases, and all are suspect pathogens. Therefore, the finding of any salmonellae, whether in foods, animal or human carriers, sewage waters or in the polluted environment, calls for elimination in order to protect the public. An extensive review of the *Salmonella* problem in relation to sewage was done by Nero (1974).

Pseudomonas aeruginosa group--*Pseudomonas aeruginosa* is an "opportunistic pathogen" and a common cause of ear, sinus and burn infections (Forkner, 1960; Prier and Friedman, 1974). *P. aeruginosa* infections are persistent and can be serious due to its resistance to many antibiotics (Artenstein and Sanford, 1974 and Brown, 1975). Its occurrence in lower animals is not common unless such animals have frequent contact with man, e.g., domestic animals and especially young animals which are most likely to be handled by man (Hoadley and McCoy, 1968). *P. aeruginosa* is regularly found in wastewaters (Hoadley, et al., 1968).

Staphylococcus aureus group-- *Staphylococcus aureus* is also an opportunistic pathogen which is capable of producing a variety of infections and a form of food poisoning (Elek, 1959; Prier and Friedman, 1974). While it was originally susceptible to penicillin and many antibiotics, it is prone to develop resistance, and such resistant populations are now found in many hospital-acquired infections. Recently, several other species of *Staphylococcus* associated with the human body have been reported (Kloos and Schleifer, 1975 and Schleifer and Kloos, 1975). Not all of these species are yet known to be pathogens. Coagulase positiveness is generally indicative of pathogenicity in *S. aureus*, but cases of coagulase negative staphylococcal infections are known (Quinn, et al., 1965). It is, therefore, probable that the familiar *S. aureus* is one of a group of staphylococci of potential pathogenicity. The approach taken for this investigation was to enumerate all *Staphylococcus* colonies on a specific growth medium (Kloos and Schleifer, 1975, Schleifer and Kloos, 1975) and to confirm true *S. aureus*, if present by the coagulase test. The situation is thus comparable to that for the total coliform (TC) and fecal coliform (FC) bacteria in wastewater.

Laboratory and Field Studies

Materials and Methods--

Samples and handling before testing--Bacteria are living cells with potential for either growth or death after sampling, thereby differing from viruses. Special precautions are needed if bacteriological counts are to represent the bacterial populations at the time and point of sampling. Error can occur in two ways: 1) growth or die-off after sampling or 2) failure to detect the target bacteria. The latter is a particular problem in the counting of certain pathogens, which are always few in number and generally unable to compete in the massive bacterial population in sewage. In some cases, the actual numbers of the target counts are important in order to compare with national standards, e.g., the fecal coliforms (FC). More often in these investigations the numbers are important only to calculate bacterial density (numbers per ml or per 100 ml) before and after treatment so as to determine the effectiveness of certain treatment techniques. Thus, accurate bacteriological enumeration can only be accomplished through proper sampling, preservation, rapid transport and prompt testing in the laboratory.

If the bacteriological sample is to reflect the actual bacterial count of the waste in question, the sample must be taken in a sterile container and in a manner to avoid contamination, i.e., aseptically. Wide-mouthed polypropylene bottles of 500 or 1000 ml capacity are convenient; they are autoclavable and non-breakable in the field. Their screw top closure is also leakproof and is a safeguard against spreading of possible pathogens in the laboratory.

The method of sampling depends on the nature and accessibility of the sewage to be tested. The choice of type of sample is determined in part by the conditions and purpose of testing.

Flow composited samples were preferred where feasible, since they provide more uniformity and thus, minimize the need for replicating. In general, the composite samples were taken from a holding vessel (originally sterile), to which a portion of the sewage being treated was diverted continuously or periodically over a 24-hr time. Such a composite sample was preferable to a grab sample.

Grab samples were often taken to represent aerobic unit mixed liquors or septage. In such cases, care was taken to well mix the liquid before sampling so that suspended solids were uniformly distributed. However, non-uniformity of grab samples is inevitable, since the solids themselves are heterogeneous in nature, size and settling characteristics. Large volumes of sample, mechanical blending in the laboratory and replication of tests (even of the sub-samples for each laboratory test) were required to minimize error in results based upon grab sampling.

Grab samples of septic tank sludge are sometimes desired. They can be taken by a sterile bottle, lowered to the point of sampling and then opened, or by a lake mud sampler which is also opened in situ. The mud sampler was used which is made by the Martek Co. and consists of a cone-shaped chamber covered by a larger diameter disc. When the apparatus is in place, a rod pushes the cone down and allows the sample to enter. Raising the cone closes the chamber and protects the sample from contamination as the apparatus is withdrawn. The sample was transferred by sterile spatula to a sterile plastic bag, such as the Whirl-Pak type. The apparatus was rinsed in water and alcohol and flamed to sterilize before the next sample was taken.

Soil samples were sometimes taken from various points surrounding soil absorption systems. If the systems were opened by excavation, samples were taken from the clogged zone just under or at the lower sidewalls of the systems. This zone is usually only a few centimeters in thickness. Soil adjacent laterally or below the systems were sampled by exposing to the desired point (30 to 60 cm or more) by further excavation. Exposed surfaces were then sampled with a sterile spatula or trowel, and the soil promptly transferred aseptically to a sterile Whirl-Pak bag. Occasionally, a core of soil was preferred and such a core was extracted by a simple open-ended brass cylinder closed at both ends with rubber stoppers. The stoppers were removed before insertion and the outer one replaced before withdrawal while the other one promptly after withdrawal. Another convenient sampler consisted of a longer cylinder cut-away along one side with a sheath to close this opening before sampling. After withdrawal the sheath could be rolled back to allow sampling at desired points within the core.

Well-point water samples were also used in monitoring absorption fields without excavation. The well points were driven to the desired depths and left in place. When a sample was taken, water was withdrawn through sterile tubing by vacuum. The first sample drawn was discarded so that the sample taken afterward would represent the free water in the bed or ponded trench, not the stagnant water in the "well." The well-point type of sampling was often preferred to excavating, since the sites need not be disturbed.

Samples taken by any of the above methods were promptly ice-refrigerated in the field and during transportation to the laboratory. For local samples, the testing began as soon as possible, always within 24 hours. Freezing with dry ice was not done, because such a procedure causes a serious reduction in the counts (Calcott, et al., 1975).

Analytical Methods--Standard Methods (1971) recommendations are helpful, but not always strict enough or broad enough for purposes of specific investigations. Therefore, in some instances, the "standard method" was adapted or new procedures developed for both sample handling and analysis. A brief statement of the tests conducted is given here.

The following counts were routinely done for the bacteriological monitoring:

1. Total Coliforms (TC)
2. Fecal Coliforms (FC)
3. Fecal Streptococci (FS)
4. "Total" bacteria count (TBC)
5. *Salmonella* spp.
6. *Pseudomonas aeruginosa* group
7. *Staphylococcus aureus* group

Depending upon the need to know these counts, choices were made from the list. Usually the TC, FC, and FS as indicator groups were most useful. Usually tests also were made for one or more of the pathogens, most often *Pseudomonas* for reasons given later. In general, the counts were done on the MPN (Most Probable Number) basis, a practice approved in Standard Methods (1971). For some tests, the conventional plate count was preferred, especially when a selective growth medium would allow selection of colonie for confirmation of the bacterial type. The newer more rapid Membrane Filter (MF) technique is becoming popular, but it is most applicable to clear liquid samples. With turbid samples, clogging of the filter with slime, sludge or soil is a problem. If high counts of a specific bacteria are expected, dilution of the sample in sterile, peptone-buffered water blanks before filtration is helpful, but not always feasible.

Table C-1 summarizes the tests used.

TABLE C-1. RECOMMENDATIONS FOR SAMPLING AND TESTING OF SEWAGE SAMPLES.

Sample	Test By Counts For:						
	TBC	FC	TC	FS	P. aerug.	Staph.	Sal.
Treatment Unit Influent (Composite or Grab)	Pl	Mf	Mf	Mf or Pl	MPN	Mf (En, Pl)	MPN (En, Pl)
Treatment Unit Effluent (Composite or Grab)	Pl	Mf	Mf	Mf or Pl	MPN	Mf (En, Pl)	MPN (En, Pl)
Grey Water (Grab)	Pl	MPN	MPN	Pl	MPN	Mf (En, Pl)	MPN (En, Pl)
Septage (Grab)	Pl	Mf	Mf	Pl	MPN	Mf (en, Pl)	MPN (En, Pl)
Septic Tank Sludge (Grab)	Pl	MPN	MPN	Pl	MPN	Pl	MPN (En, Pl)
Clogged Zone and Adjacent Soil (Soil)	Pl	MPN	MPN	Pl	MPN	En, Pl	
Well Point (Grab)	Pl	MPN	MPN	Pl	MPN		

Pl = Plating Mf = Membrane filter MPN = Most Probable Number En = Enrichment

Results of Field and Laboratory Studies--

Bacterial quality of various wastewater effluents--In fresh raw wastewater, the kinds and numbers of bacteria are predominantly those of intestinal origin. As the waste passes through treatment, conditions change and new types of bacteria flourish and compete with the intestinal bacteria. The microflora of sewage, therefore is a complex mixture of bacteria growing and dying in response to their environment. In the aggregate, they are capable of biodegradation of numerous chemical compounds in the waste. In this respect, they differ from the viruses which are merely carried along without multiplication after leaving the body.

Anaerobic treatment--The septic tank is anaerobic with the majority of the functioning bacteria being facultative, capable of growth with or without free oxygen. Because the wastewater is a high protein/low carbohydrate substrate, proteolytic bacteria becomes dominant. The intestinal indicator types are at a disadvantage, since they are fermentative, growing preferentially when carbohydrate is available. Their numbers may decline during septic tank treatment, but they are still present in significant numbers in the effluent (See Table C-2). In addition, the pathogens, *Staphylococcus aureus* and *Salmonella* spp. have been isolated in septic tank effluent (See Table C-3).

Aerobic treatment--Aerobic treatment of wastewater offers a more advanced degree of degradation where aerobic bacteria become dominant. These also compete with intestinal indicator bacteria and pathogens but as Tables C-2 and C-3 indicate, high numbers can still survive through aerobic treatment units and sand filters. *Ps. aeruginosa* was found in higher numbers in the mixed liquor of extended aeration units than in septic tanks, which indicates that conditions are more suitable for its survival or growth in aerobic tanks (Ziebell, et al., 1975b).

Thus, it is evident that if soil is to be used for ultimate disposal, it must be capable of removing the remaining harmful bacteria.

Bacterial removal from wastewater by soil--It is well known that soils have a tremendous capacity to remove bacteria from wastewater. However, it is difficult to specify the depth of soil through which the wastewater must percolate to remove potentially hazardous levels of bacteria. Soil type, temperature, and pH; bacterial adsorption to soil and soil clogging; soil moisture and nutrient content; and bacterial antagonisms are factors which affect bacterial survival and movement.

Laboratory studies--To study bacteria removal from septic tank effluent by different soils, eight soil columns were set up for study under controlled laboratory conditions (Ziebell, et al., 1975a). The columns were 10 cm in diameter and 60 cm in depth. Columns 1 through 4 were hand packed with sand from the C horizon of Plainfield loamy sand. Columns 5 through 8 combined undisturbed cores of the A2 and B21 horizons of Almena silt loam which had been taken in the field and coated with paraffin to facilitate handling (Daniel and Bouma, 1974). The particle size distribution of these soils and other relevant physical characteristics are presented in Table C-4. The sand had a single grain structure, and the Almena silt loam had a prismatic structure (2 to 5 cm wide and 5 to 10 cm high, relatively compact natural

TABLE C-2. REPRESENTATIVE BACTERIOLOGICAL COUNTS AT VARIOUS STAGES OF TREATMENT (after Ziebell, *et al.*, 1974b).

Sample Type	Total Bacteria no/100 ml	Fecal Coliforms no/100 ml	Total Coliform[1] no/100 ml	Fecal Streptococci[1] no/100 ml	P. aeruginosa[1] no/100 ml
Septic Tank[2] mean 95% conf. intr.	$34x10^7$(88)[6] $25x10^7$-$48x10^7$	$42x10^4$(84) $29x10^4$-$62x10^4$	$34x10^5$(91) $26x10^5$-$44x10^5$	3800 (97) 2000-7200	10,000 (13) 1900-54,000
Extended Aeration Units mean 95% conf. intr.	$18x10^7$(80) $7.4x10^7$-$28x10^7$	11,000 (67) 7400-15,000	$11x10^4$(74) $7.1x10^4$-$16x10^4$	3300 (70) 2300-4800	2500 (7) 240-26,000
Sand Filter[4] w/ septic tank influent mean 95% conf. intr.	$4x10^7$(15) $2.2x10^7$ $7.2x10^7$	4200 (16) 1000-17,000	13,000 (14) 3100-53,000	94 (16) 40-230	---------
Sand Filter[5] w/ extended aeration mean 95% conf. intr.	$1.1x10^7$(25) $0.7x10^7$-$1.6x10^7$	3100 (24) 1800-5200	14,000 (21) 8200-23,000	730 (26) 380-1400	590 (4) 120-700

[1] Log normalized data

[2] Includes data from 5 tanks

[3] Includes data from 3 units

[4] e.s. = 0.45, u.c. = 3.0; loading 64-128 cm/day (16-32 gpd/ft^2)

[5] e.s. = 0.19, u.c. = 3.3; loading 16-24 cm/day (4-6 gpd/ft^2)

[6] number of samples

TABLE C-3. DETECTION FREQUENCY FOR STAPHYLOCOCCUS AUREUS AND SALMONELLA SPP. IN WASTEWATER FROM FIELD SITES (after McCoy and Ziebell, 1976).

Site	No. Occupants	Effluent Sampled	*Staphylococcus aureus*		*Salmonella* spp.	
			No. Samples	No. Positive	No. Samples	No. Positive
A	3	Septic Tank	1	0	7	0
B	3	Septic Tank	1	0	6	3
C	6	Extended Aeration Unit	7	1	9	1
E	4	Septic Tank	14	1	6	0
F	3	Septic Tank	6	0	3	0
G	5	Extended Aeration Unit	8	1	10	0
H	4	Extended Aeration Unit	16	0	8	0
I	7	Rotating Biological Disk Unit	5	3	1	0
J	13	Septic Tank	4	0	-	-
K	5	Septic Tank	1	0	1	0
O	5	Septic Tank	-	-	2	0
P	4	Septic Tank	-	-	2	1
Total			63	6	55	2
Percent Positive			9.5		3.6	
Range of Positives			10-1000/100 ml		3.4->20/100 ml	

TABLE C-4. PHYSICAL CHARACTERISTICS OF SAND FROM THE C HORIZON OF THE PLAINFIELD LOAMY SAND AND OF THE A1 AND B21 HORIZONS OF THE ALMENA SILT LOAM (Ziebell, *et al.*, 1975a).

	Particle size distribution USDA classification (%)									Texture	Particle density (gr/cm^3)	Bulk density (gr/cm^3)
	Clay	Silt			Sand							
		fine	medium	coarse	very fine	fine	medium	coarse	very coarse			
C horizon Plainfield	--	--	--	1.00	0.11	0.63	6.68	86.0	6.2	coarse sand	2.63	1.67
A2 horizon Almena	21	11.6	30	25	10.5	1.2	0.6	0.1	--	silt loam	2.26	1.45
B21 horizon Almena	27	6.1	18	36	12.7	0.1	0.1	--	--	silt loam	2.22	1.60

soil aggregates or peds, separated by planar voids). The sand columns had one tensiometer at a depth of 5 cm below the sand surface. Six air exchange tubes were placed at various levels in the column walls to ensure aerobic conditions in the sand (Magdoff, et al., 1975a). The silt loam columns had four tensiometers at depths of 5, 10, 20 and 30 cm below the surface (Daniel and Bouma, 1974). All columns were covered with vented plexiglass caps to restrict drying.

The columns were subjected to different dosing regimes of septic tank effluent at various temperatures (See Table C-5). The sand columns received daily dosages of 5 and 10 cm of effluent. These loadings were chosen on the basis of field monitoring studies. The infiltration rate through the clogged layer in sands is approximately 5 cm/day (1.2 gpd/ft^2) (Bouma, et al., 1972). This loading rate, therefore, is applicable to soil disposal systems, including mound systems, in the field to allow continued functioning even when clogged (Bouma, et al., 1974b; Bouma, 1975). However, irregularities in liquid distribution may result in local overloading and the higher loading rate was, therefore, also included in the experimental plan to investigate its effect on purification.

TABLE C-5. EXPERIMENTAL DESIGN FOR COLUMN STUDIES INVESTIGATED REMOVAL OF PATHOGENS BY SOIL (Ziebell, et al., 1975a).

Column	1	2	3	4	5	6	7	8
Soil	...Plainfield loamy sand...				...Almena silt loam...			
Temperature (°C)	25	25	5	5	25	25	25	25
Loading (cm/day)	10	5	10	5	cont. ponding	cont. ponding	1	1

Soil disposal systems must function during the entire year, under variable temperature conditions. In situ measurements of soil temperatures in mound systems were found to range approximately 5 to 25°C suggesting the temperatures to be used in these experiments (Bouma, et al., 1974c; Bouma, 1975).

The type of silt loam tested has a low hydraulic conductivity at saturation, generally less than 4 cm/day (1 gpd/ft^2) and sometimes even lower (Daniel and Bouma, 1974). A dosing rate of 1 cm/day (0.24 gpd/ft^2) applied to columns 7 and 8 permitted complete infiltration within a day, thereby allowing the infiltrative surface to be exposed to the air so that aerobic decomposition of clogging compounds could occur. Columns 5 and 6 were continuously ponded by maintaining a constant head above the infiltrative surface. The ponded columns simulated conditions where the loading rate exceeded the capacity of the soil to conduct the liquid downward.

Travel times of liquid were determined in the columns with a 300 mg/L KCl solution as a tracer, used as described by Converse, et al., (1975a).

The calculated retention times versus loading rates are given in Table C-6.

Septic tank effluent was obtained weekly from a single family residence and stored at 4°C until used. Since it contained relatively low numbers of fecal coliforms and fecal streptococci, additions of each were made. An isolate of each indicator organism was obtained from the sewage incubated for 20 to 24 hours in shake flasks with Nutrient Broth (Standard Methods, 1971), and added to the septic tank effluent to obtain average concentrations of 5.1×10^6 FC/100 ml and 7.3×10^6 FS/100 ml. Sewage, thus fortified, was prepared every two days and stored at 4°C, with insignificant die-off occurring in this period.

Staphylococcus aureus (*S.a.*) generally was not present in the septic tank effluent from this site. An isolate of this organism (*S.a.* FDA 209) was also grown in nutrient broth and added to obtain initial concentrations of 10^4/100 ml of septic tank effluent; this number is similar to those found in some septic tank effluents (Ziebell, et al., 1975b). However, rapid die-off of this organism occurred, dropping the counts by approximately 2 logs during the 2 day storage. Nevertheless, the numbers of *S.a.* applied to the columns were still in a realistic range for *S.a.* in sewage, i.e., 10^2-10^4/100 ml.

Pseudomonas aeruginosa (Ps.a.) was present and survived in the septic tank effluent; no attempt was made to alter its numbers. The average concentration was 1400/100 ml.

Before the start of experiments, tap water was applied to wet all columns at the respective loading volumes for three days prior to initial septic tank effluent application. The prepared fortified septic tank effluent was then applied once daily to all columns except #5 and #6 which were kept ponded by using Mariotte bottles filled every two days with fortified septic tank effluent. Samples were collected in sterile polypropylene bottles with screw cap lids attached by tubes to the base of the column.

The numbers of bacteria in the fortified septic tank effluent and in column effluents were monitored regularly over a period of 200 days. Soil moisture tensions and column effluent volumes were recorded prior to each dosing. Occasionally, soil moisture tensions were recorded periodically throughout a day to establish diurnal column performance and to relate changes in the soil moisture recovery pattern in terms of moisture conditions within the column as the experiment progressed. For example, these data were helpful in judging whether clogging was developing.

Figure C-1 and C-2 compare results from Columns 1 and 2 packed with Plainfield loamy sand: both at 25°C with Column 1 receiving 10 cm and Column 2 receiving 5 cm of effluent per day. Both columns initially removed bacteria effectively, but after the first 5 to 10 days, they began to release FC in their effluents. During the first 100 days of the experiment, the number of effluent FC reached a plateau of approximately 3×10^5 FC/100 ml from Column 1 and 10^3/100 ml from Column 2, as compared to the influent waste containing 5.1×10^6 FC/100 ml. While these are unacceptably high for a final effluent, they still represent removals of 94.1 and 99.98% respectively.

TABLE C-6. RESULTS OF CHLORIDE TRACER STUDIES FOR DETERMINING TRAVEL TIMES OF LIQUID AT DIFFERENT DOSING REGIMES IN SAND AND SILT LOAM COLUMNS (Ziebell, et al., 1975a).

Column No.	Loading rate (cm/day)	K_{sat} (cm/day)	First chloride appearance		Appearance of 150 ppm chlorides		Appearance of 300 ppm chlorides		Volume of water-filled pores at equilibrium (cm^3)
			Time	Cumulative outflow (cm^3)	Time	Cumulative outflow (cm^3)	Time	Cumulative outflow (cm^3)	
1 and 3	10	500	1.5 hrs	618	23 hrs	800	25 hrs	1018	720
2 and 4	5	500	26 hrs	675	47 hrs	780	50 hrs	1025	720
5	ponded	5.2	5 days	1600	8.7 hrs	2782	13 days	4160	1700
6	ponded	1.0	15 days	1545	24 days	2472	33 days	3400	1600
7a	1	7.9	10 days	700	19 days	1330	33 days	2310	1455
7b	1		5 hrs	70	8 days	560	18 days	1260	1455
8	1	2.4	10 days	640	21 days	1344	35 days	2240	1575

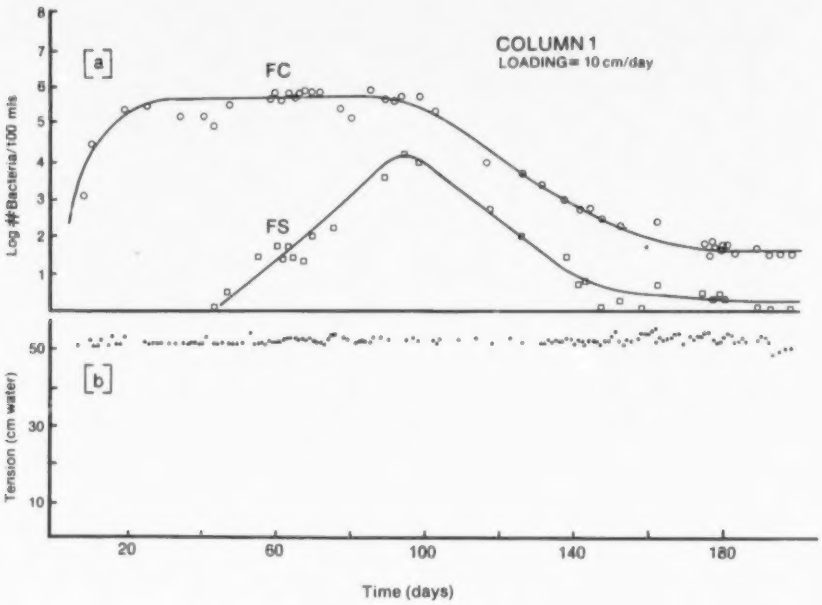

Figure C-1. a. Bacterial data from Column 1 (Plainfield ls, 25° C); FC = Fecal coliforms, FS = Fecal streptococci.
b. Soil moisture tension 5 cm below the infiltrative surface before daily dosing (Ziebell, et al., 1975a).

Fecal streptococci appeared in the effluent from Column 1 (high loading) on day 48 and reached a peak of 13,000/100 ml on day 95. Fecal streptococci, however, were never detected in the effluent of Column 2 (low loading). At approximately 100 days and thereafter, the effluent FC counts of both Columns 1 and 2 and the FS of Column 1 began to decline until the end of the 200-day experiment.

Pathogen removals were also detected through the columns. *Pseudomonas aeruginosa* was detected in three effluent samples from Column 1 out of 19 tested: 23/100 ml on day 91, 3/100 ml on day 96, and 3/100 ml on day 128. *Staphylococcus aureus* was not found in any of the 15 Column 1 samples tested, although 200 to 600 ml samples were analyzed. These pathogens were never detected in the outflow from Column 2, although sample volumes of 30 to 50 ml for *Ps.a.* and of 100 to 350 ml of *S.a.* were tested.

The performance of these columns can also be judged by the volume of outflow and the moisture tension characteristics. The hydraulic data for these columns showed sustained equilibrium conditions in terms of volume of effluent collected daily and moisture tension stability (~50 cm of water). Column 1, after 100 days, showed some lag in recovery of tension after each application, thus suggesting some flow impediment or early stages of clogging (Figure C-1b). Column 2 tensiometer data did not show evidence of this condition (Figure C-2b).

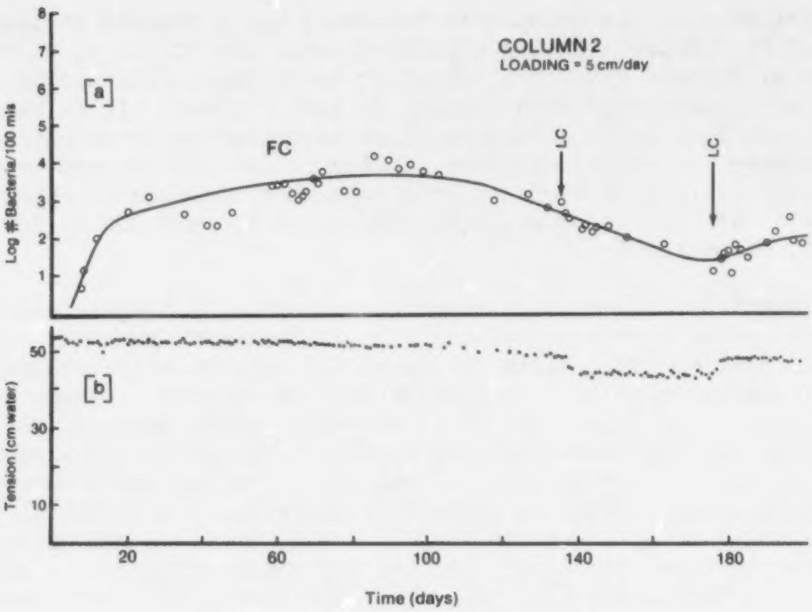

Figure C-2. a. Bacteria from Column 2 (Plainfield ls, 25°C); LC = Loading change, i.e., first change from one 5 cm/day dose to three applications per day of 1.7 cm each, and the second change restoring the original dose regime.
b. Soil moisture tensions 5 cm below infiltrative surface before daily dosing (Ziebell, et al., 1975).

Toward the end of the experiment, a change in the dosing regime was made for Column 2 in order to test multiple dose application. On day 137, the same daily quantity of 5 cm was applied, but it was divided into three 1.7 cm doses. The first was applied in the morning, and the remaining two at approximately 4-hr intervals. There was only a slight reduction in bacterial counts and on day 176 the original once-a-day rate of 5 cm/day was restored.

Columns 3 and 4 also packed with Plainfield loamy sand were maintained at 5°C. They presented a somewhat different pattern including a drastic change when a malfunction of the refrigeration occurred on day 48. (See Figures C-3 and C-4). Thus, the data should be analyzed with respect to the temperature effect, remembering the at 5°C bacterial growth and metabolism are very slow.

In the first days of operation, both Columns 3 and 4 removed at least 4 log numbers of FC, before allowing a peak of about 10^3 FC/100 ml to pass. The peak period of release was short, about 10 to 20 days, followed by rapid decline, which *coincided with ponding* in both columns. It is reasonable to assume that this ponding resulted from accumulation of organic compounds which were not decomposed very rapidly at the low temperature. During this ponded period, tensions in both columns stabilized at approximately 30 cm, and the volumes of effluent dropped to 400 and 200 ml for Columns 3 and 4, respectively.

The refrigeration failure, which resulted in a rise in temperature on day 48, caused unponding with a consequent rise in bacterial numbers in the effluents, high tensions after daily drainage, and larger daily volumes of effluent, corresponding with the total daily applied volumes. Column 4 received lower dosages and, therefore, could probably remain unponded for a longer time. Both columns, when restored to 5°C, returned to the ponded state and so remained for the duration of the study, during which Column 3 passed very little liquid (about 40 ml/day) or bacteria (approximately 5 FC/100 ml). Column 4 (originally the lightly loaded one), on the other hand, functioned better in terms of liquid outflow, but still poorly in terms of bacterial removal. The higher FC counts in Column 4 effluent (as compared to Column 3) are a reflection of shorter liquid retention times. Effluent volumes from Column 4 of 100 to 200 ml/day can be associated with approximate retention times of 3 to 4 days, while liquid entering Column 3 would require approximately 12 days (at a flow rate of 40 ml/day) before leaving the column. Associated with these outflows, moisture tensions for Column 3 remained between 35 and 41 cm, while tensions in Column 4 reached 43 cm, then dropped to 22 cm with the erratic outflow volumes and FC remaining at the high 10^3/100 ml. Gas bubbles were observed between the sand grains in the clogged zone of Column 4, and disruptions resulting from their release were believed to cause these irregularities.

Although FC were present in the effluents of Columns 3 and 4, these columns functioned very efficiently in removal of FS and the pathogens, *Pseudomonas aeruginosa* and *Staphylococcus aureus*. Fecal streptococci were found in only 2 of 21 samples of Column 3 (3/100 ml on day 9, and 58/100 ml on day 50) and in only 1 of 32 samples from Column 4 (1/100 ml on day 62). *Pseudomonas aeruginosa* was present in 2 of 18 samples from Column 3 (3/100 ml on days 42 and 45) and in only 1 of 20 samples from Column 4 (3/100 ml on day 94). *Staphylococcus aureus* was not detected in either column effluent (30 samples analyzed).

In summary, data for the sand columns indicate that only 60 cm of sand can remove large numbers of fecal indicators and pathogens. Flow regime and soil temperature clearly affect the removal process, in part by inducing early soil clogging at low temperatures. Removal below normal detection levels was generally not achieved, especially during the early weeks of operation of the columns (in this study, the first 100 days). This first period, before clogging occurs, can be a critical phase of operation. During this time, high loading rates and localized overloading may effect transport of pathogenic organisms deep into the soil. A newly constructed septic tank disposal system was implicated as the source of well water

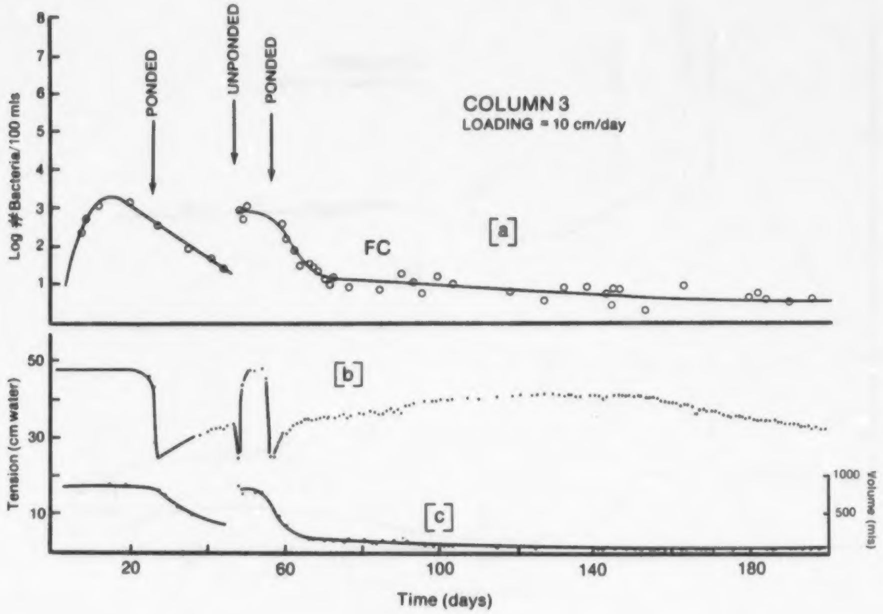

Figure C-3. a. Bacterial and physical data from Column 3 (Plainfield ls, 5°C: PD = continuous ponding after the indicated time, UPD = unponded conditions after the indicated time.
b. Moisture tension 5 cm below the infiltrative surface.
c. Volume of column effluent collected (Ziebell, et al., 1975a).

contamination causing 60 cases of gastroenteritis in an Illinois State Park (Morbidity and Mortality, 1972). The septic tank system had been installed at the required distance from the well as specified in the local code, however, during installation, groundwater was observed at 10 feet from the soil surface and the well water was reported to be turbid for a short period after installation.

The retentive power of the columns improves as bacterial films build up on the sand surfaces. However, such columns can allow escape of FC and FS and pathogens. Therefore, for safety, more than 60 cm of sand is required.

Low dosing rates significantly enhanced removal of fecal indicators and pathogens, indicating that field system overloading, as represented by the columns loaded at 10 cm/day, should be avoided.

Results of the chloride tracer study with silt loam columns are reported in Table C-6. Columns 5 through 8 show a significant difference between the ponded Columns 5 and 6 and the columns which received daily dosages of only 1 cm, i.e., Columns 7 and 8. Five and 15 days were required for displacement of liquid present in Columns 5 and 6, respectively, as evidenced by the first appearance of chlorides. The longer retention time for Column 6 was

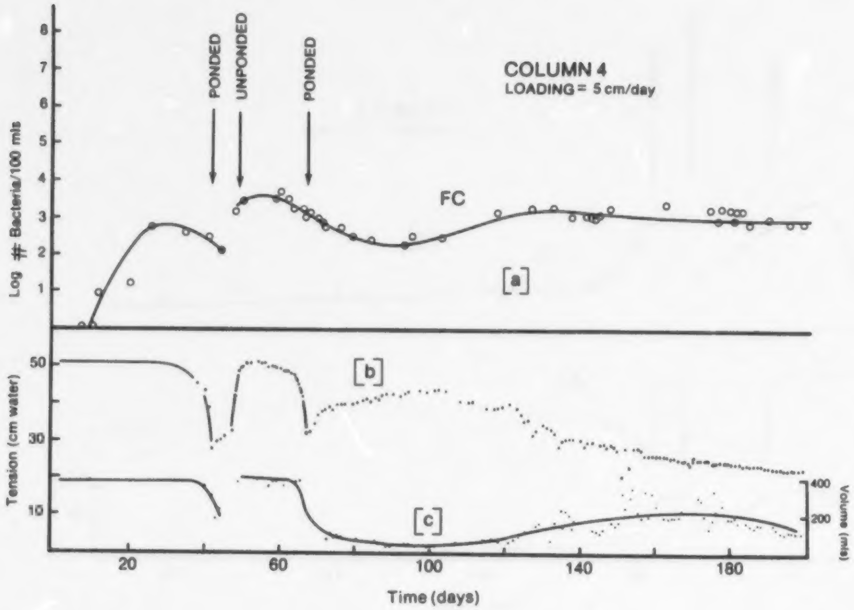

Figure C-4. a. Bacterial and physical data from Column 4 (Plainfield ls, 5°C): PD = continuous ponding after the indicated time, UPD = unponded conditions after the indicated time.
b. Moisture tension 5 cm below the infiltrative surface.
c. Volume of column effluent collected (Ziebell, et al., 1975a).

was due to its low hydraulic conductivity at saturation. The saturated hydraulic conductivity is controlled by the size and continuity of few, but relatively large, planar and tubular voids in the soil. These are the only pores conducting significant quantities of liquid at saturation since water moves very slowly through the soil aggregates (Bouma and Anderson, 1973).

Columns 7 and 8 received a daily volume of liquid at a rate less than the saturated hydraulic conductivity. Therefore, these columns drained successfuly in one day, as evidenced by an unponded surface several hours before the next liquid addition. The total volume of water-filled pores after drainage, i.e., at equilibrium, was 1455 and 1575 cm^3 for Columns 7 and 8, respectively. However, chlorides appeared in the column effluent after passage of only 700 and 640 cm^3 of liquid, or about half the volume of liquid present at equilibrium.

This can be interpreted by considering the nature of flow processes in aggregated soils. Immediately after dosing, the large air-filled pores fill with liquid and conduct this liquid considerable distances, depending on pore-continuity. The liquid present inside the aggregates is bypassed.

If such short-circuiting extends deep enough it could have implications on sewage purification. The retention data reported for Columns 7a and 8 in Table C-6 were determined at the start of the experiment, indicating relatively long retention times of 10 days. However, changes occurred in the pore structure of Column 7. Interconnection of larger pores, perhaps caused by faunal activity in these undisturbed cores, resulted in significant short-circuiting. After the bacteriological experiments were completed, chloride tracer studies were again conducted on Column 7. Results presented in Table C-6 (Column 7b) show that chlorides appeared in the effluent within 3 hours, after only 70 cm^3 of liquid had passed through the column. Field measurements have been reported indicating similar phenomena in undisturbed soils below seepage systems (Bouma, et al., 1974b; Bouma, 1975).

Bacteriological analyses of effluents from Column 5 and 6 indicated excellent removal of fecal indicator bacteria from sewage after continuous ponding occurred. Two of 22 effluent samples from Column 5 contained 10 to 13 FC/100 ml and of these only 13 contained FS (2/100 ml). *Pseudomonas aeruginosa* was found in four of the 13 samples, with highest numbers occurring after day 100 (23, 15, $\geq$ 240, and $\geq$ 24,000/100 ml on days 100, 104, 143 and 148, respectively). These data indicate conditions within this column were favorable for survival and possible growth of this bacterium. *Staphylococcus aureus* was not detected in 14 samples analyzed.

Even better results were found for Column 6. An average of 15 samples were tested for each organism (FC, FS, *Ps.a.* and *S.a.*), none of which contained detectable numbers of these bacteria.

Fecal coliforms, FS and *Pseudomonas aeruginosa* were found in the effluent of Column 7, as indicated in Figure C-5. The concentrations of these bacteria reached 83,000 FC/100 ml, 100 FS/100 ml and greater than 2 x 10^6 *Ps.a.*/100 ml in Column 7 effluent on day 91. The numbers of *Pseudomonas aeruginosa* were often greater than those of the influent sewage, indicating (as in Column 5) conditions favorable for survival and potential growth. The large number of fecal bacteria in the effluent implied short-circuiting between soil aggregates or through channels formed by roots or worms as discussed earlier. On day 93, the loading was changed to 3 mm/day to determine if a reduction of the loading rate would reduce the amount of liquid available for vertical flow along cracks and channels, by allowing lateral capillary forces to pull the liquid into the aggregates. Movement of sewage through the aggregates would be expected to result in longer liquid retention, more contact of sewage bacteria with soil particles and better purification. The results confirmed this hypothesis. After the loading change, fecal indicator bacteria were undetectable in the effluent within 30 days, and similarly *Pseudomonas aeruginosa* within 90 days. *Staphylococcus aureus* was never found in Column 7 effluent.

Fecal coliforms and FS were also found in the effluent of Column 8 during the first 40 days when unsaturated flow conditions existed (See Figure C-6). However, the flow rate through this column gradually decreased below 1 cm/day, and ponding resulted with unsaturated flow below the infiltrative surface as indicated by tensiometer data (See Figure C-6). At this point, Column 8 was

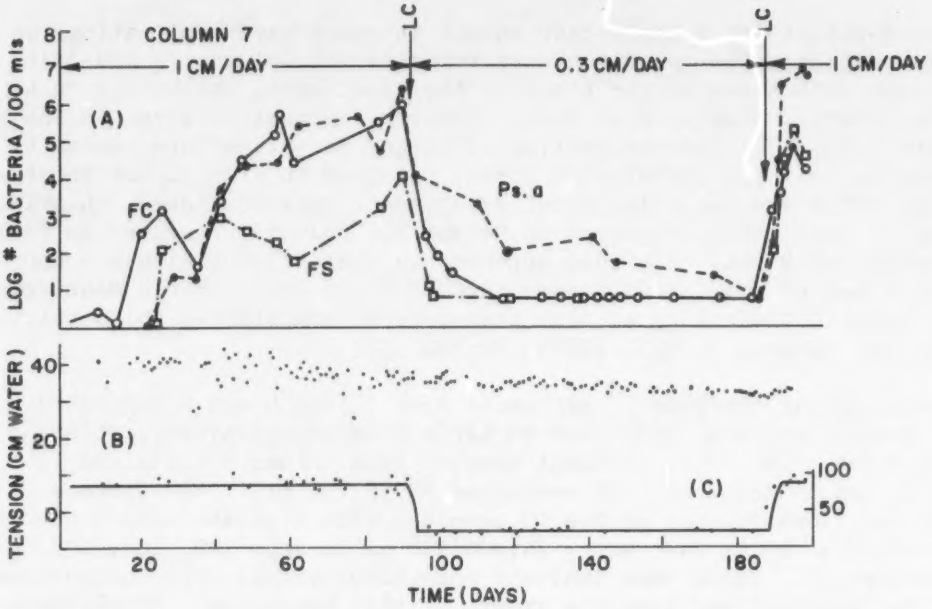

Figure C-5. a. Bacterial data from Column 7 (Almena sil, 25^{o}C): FC - fecal coliforms, FS - fecal streptococcus, *Ps. a.* = *Pseudomonas aeruginosa*.
b. Moisture tensions, 5 cm below the soil surface, prior to daily dosing.
c. Volume of column effluent (Ziebell, et al., 1975a).

operating under conditions similar to Columns 5 and 6. Indicator organisms were no longer detected in the effluent. *Pseudomonas aeruginosa* and *Staphylococcus aureus* were never found in 12 and 14 respective samples from this column.

In summary, data derived from the silt loam columns indicate that 60 cm (24 in) of a slowly permeable silt loam soil can remove fecal bacteria very effectively under certain flow regimes. However, the heterogeneous pore structure of these aggregated clayey soils affects removal efficiency. Short-circuiting of effluent through large air-filled pores resulted in rapid movement of fecal indicators for considerable distance, as demonstrated in Columns 7 and 8. These observations have practical implications for construction of on-site disposal systems in slowly permeable soils having seasonally high groundwater tables. Bacterial contamination of groundwater from septic tank-soil absorption fields under such conditions in similar soil has been shown by others (Viraraghaven and Warnock, 1973). The theoretical pore continuity patterns in clayey soils indicate that the large pores, through which short-circuiting occurred in this study, would not extend into the soil indefinitely, and problems of groundwater contamination are unlikely

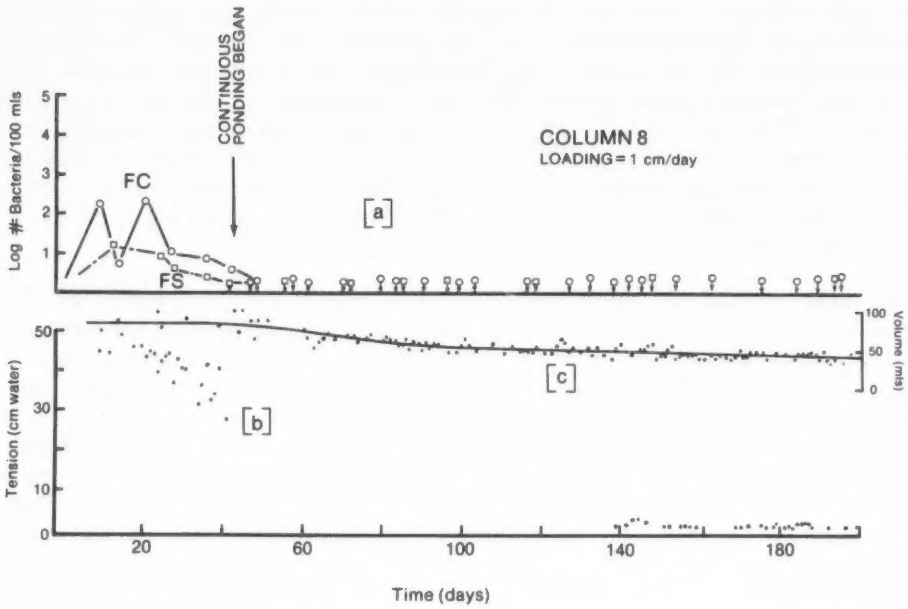

Figure C-6. a. Bacterial data from Column 8 (Almena sil, 25°C): FC = fecal coliforms, FS = fecal streptococcus.
b. Moisture tensions, 5 cm below soil surface prior to daily dosing.
c. Volume of column effluent (Ziebell, et al., 1975a).

in slowly permeable soils having deep groundwater tables (Bouma and Anderson, 1973).

Removal of high groundwater tables by draining, with surface discharge of the drainage water, is being used as a procedure to improve on-site conditions for disposal of septic tank effluent (Bouma, 1975). However, short-circuiting of effluent could lead to pollution of the water in these drains and to surface water contamination following drainage discharge. One such field system has been investigated, and, it was found that liquid in the curtain drain had a high content of fecal indicator bacteria (Ziebell, et al., 1973). This problem is particularly relevant because drains in slowly permeable soils have to be deep and within a few feet of seepage trenches to function properly from a hydraulic standpoint.

Field studies--The field studies included investigations of 19 conventional subsurface soil disposal systems (Bouma, et al., 1972). Some of the systems were sampled at different times of the year, thus, they reflect in some degree the seasonal variation in biological activity and consequent problems. The general purpose of the bacteriological investigation was to monitor the number of coliform and enterococcus organisms in septic wastes, in soil samples taken at various points in drainage fields, and in test wells located around a number of such systems. A total bacterial count (TBC) was also obtained in order to evaluate the interaction of the general soil microflora and the sewage microflora in the area. The enterococcus count was assumed to be equivalent to the fecal streptococci (FS) as *Streptococcus faecalis*. These counts, and total (TC) and fecal (FC) coliform counts gave an indication of bacterial movement and density in the field system. Movement of any of these pollution-indicating bacteria to the surface of the soil or to the groundwater was considered a priori evidence of unsafe conditions, i.e., failure of the system.

Detailed data on the bacterial populations for a single system in Plainfield loamy sand illustrates the general characteristics of all 19 investigations. Additional data from other systems studied may be found in a report (Bouma, et al., 1972). The counts on the septic effluent entering the seepage field were typically high for all of the bacterial tests. All three of these pollution bacteria were rapidly removed by soil adsorption below the trench (See Figure C-7), as were a great many of the general bacteria of the septic effluent, as shown by a drastic drop in the TBC count. The population in the seepage bed is reduced within 30 cm below or to the side of the trench to about the level of the population in control soil. The abrupt drop in numbers occurs between the clogged zone and 30 cm horizontally or vertically.

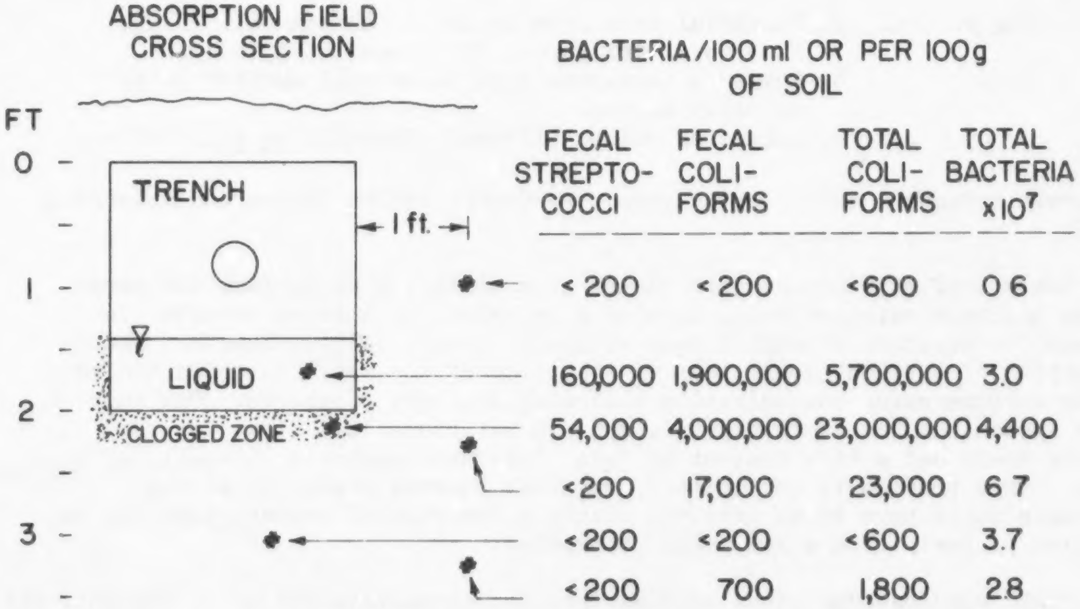
ABSORPTION FIELD CROSS SECTION
BACTERIA/100 ml OR PER 100g OF SOIL
FT
0
1
2
3
TRENCH
1 ft.
LIQUID
CLOGGED ZONE
FECAL STREPTO-COCCI
FECAL COLI-FORMS
TOTAL COLI-FORMS
TOTAL BACTERIA x10^7
<200 <200 <600 0.6
160,000 1,900,000 5,700,000 3.0
54,000 4,000,000 23,000,000 4,400
<200 17,000 23,000 6.7
<200 <200 <600 3.7
<200 700 1,800 2.8

Figure C-7. Cross-section of an absorption field in Plainfield loamy sand with typical bacterial counts at various locations (Ziebell, et al., 1975b).

A darkening due to the presence of iron sulfide defines the clogging zone, which implies anaerobic conditions within the zone. However, the bacteria within the zone are not necessarily obligate anaerobic but are predominantly facultative types functioning anaerobically (Bouma, et al., 1972). The coliforms, fecal streptococci and many of the TBC bacteria are facultative. From inspection of bacterial counts in the clogged zone, it is not possible to say whether the bacteria living there result from trapping by adsorption or from growth. Probably both processes occur, since nutrients, moisture, pH and temperatures are favorable. One bit of evidence for growth of bacteria in the zone is found in the high numbers of pseudomonads (*Pseudomonas* spp., including *P. aeruginosa* but also the *P. fluorescens* group which is common in soils but not in the feces). These pseudomonads are found in even greater numbers in the adjacent soils below and at the side of the trench/soil interface. In fact, they and certain yellow and orange *Flavobacterium* spp. and red *Serratia* spp. can comprise up to 70% of the TBC counts. The high pseudomonad count (it is as high as the low millions per gram of soil just beyond the clogged zone) is probably important in the final "purification" of the percolating water. Pseudomonads are known for the many carbon substrates they can use, e.g., proteins, fats, carbohydrates, and many more resistant carbon compounds like chitin, waxes, hydrocarbons, etc. (Stanier, et al., 1966). The pseudomonads also produce pyocyanins, fluorescein, HCN, phytotoxic factors and a number of endo- and exotoxins as yet not well defined (Artenstein and Sanford, 1974). Thus, they are probably a factor in the die-off of the indicator bacteria and pathogens in the percolating wastewater. Field studies also showed that within the first foot of soil, either downward or laterally, soil actinomycetes, bacilli, and molds begin to appear and they are very numerous in the second foot of soil (Bouma, et al., 1972). These are antibiotic producers and no doubt contribute to the die-off of pathogenic indicators. It is interesting that the TBC platings of soil taken beyond 30 cm from the infiltration surface appear typical of the soil flora, rather than of the sewage flora, which was still dominant in the first 30 cm. The striking difference is:

> Soil TBC: high proportion of gram positive bacteria, actinomycetes and molds.
>
> Sewage TBC: high proportion of gram negative bacteria, esp. pseudomonads and other yellow-orange pigmented gram negative bacteria.

Summary

While remarkable purification can be achieved in non-aggregated soil under conditions of established clogging and proper flow regime, it must be remembered that many soil conditions are less efficient in providing bacterial removal. During initial periods of operation (prior to clogging), conventional soil absorption systems do not provide ideal removals. Similarly, channeling due to voids between soil aggregates can result in movement of bacteria to depths of 60 cm (2 ft) or more in aggregated soils, especially under dry conditions. Under such conditions a deep soil profile or further treatment of effluent must be present to insure adequate purification.

THE FATE OF VIRUS IN SOIL

Background

Viruses differ fundamentally from other kinds of infectious agents. The "life-cycle" of a virus includes a transmission phase and infecting phase. Virus in the transmission phase is a small, inert particle which can only be made visible with the aid of an electron microscope. Virus in the infecting phase is a virtually formless presence inside cells of the host's body, serving as a pattern for production of more virus particles. Cells which are infected and producing virus tend to abandon their special functions in the body, and they sometimes die. If enough cells die or are diverted functionally, the host's body may undergo an abnormality called disease. Because the course of virus disease can seldom be influenced by therapy, there is a tendency to blame all intractable disease upon virus infections, even in the absence of any positive evidence. Most human hosts develop immunity and recover from virus infections despite the lack of effective therapeutic agents.

Virus in the infecting phase evokes the most concern; however, only virus in the transmission phase can be forestalled. The inert particles of the virus range upward from approximately 25 nm (one millionth of an inch) in diameter, depending upon the kind of virus. A particle consists of nucleic acid coated with protein. There may be some enzyme present, and (though not in the viruses with which we are concerned) some particles are enveloped with a lipid-containing material. The particle is significant because it may cause infection. Virus transmission may be prevented either by precluding the particle's passage from one host to another or by depriving the particle of its infectivity along the way.

Intestinal Viruses--

Not all viruses are transmissible through the environment. Many lose their infectivity so rapidly outside the host that they can only be transmitted by person-to-person contact. To be transmissible in vehicles such as water and food, viruses must maintain their infectivity through time and distance outside the host; those which can do so are principally those produced in the intestines. The intestinal viruses are shed by the infected host in feces; if the virus contaminates water or other material ingested by another person, infection may result.

Human feces do not always contain viruses because the human intestines are not always infected. In countries where sanitation is not advanced, children's intestines may harbor viruses more or less continuously during the first 5 years of life. Survivors of these first 5 years show a progressively lower incidence of intestinal virus infection.

Intestinal virus infections are less common in the United States but still tend most frequently to involve young children. There is also a seasonal trend: infections are most prevalent during the months of July, August and September, at least for the enterovirus group. A person who is infected with an intestinal virus may shed the virus in his feces for several weeks, perhaps with no sign of illness, before developing a level of immunity which terminates the infection. Immunity to that specific virus is durable, so that the person cannot usually be infected with it again. However, there are approximately 100 known types of intestinal viruses, so few persons acquire immunity to all of them. In the United States, most people's intestines are without virus most of the time, not because of immunity but because of sanitation. Americans are relatively vulnerable to viruses to which they may be exposed as a result of lapses in sanitation, as the experience of travelers to certain foreign countries seems to verify.

The overall rate of virus infection and shedding in the United States population is difficult to estimate accurately. A fairly recent survey in Seattle involved families with very young children (Cooney, et al., 1972). The overall incidence of viruses other than polioviruses in fecal specimens was 3.5 percent compared to 8.8 percent for polioviruses (presumably of vaccine origin). A little over 2 percent of fecal specimens from parents contained any virus; adults not closely in contact with young children would probably have shown a lower incidence. Adenoviruses showed a higher prevalence than coxsackieviruses or echoviruses; no reoviruses were reported. This is not typical of the distribution of kinds of enteric viruses reported to be detected in raw urban wastewater, but this may be as much a function of sampling and testing techniques as of the true relative incidence of different viruses. These findings do not provide a very firm basis for estimation, but it seems likely that not over 1 to 2 percent of stools produced in the United States contain virus at all. If all of this virus were enterovirus, one might expect a mean daily output of 10^6 plaque-forming units (PFU) per person per day, which is approximately 10^{-9}g of virus material per person per day (at 100 particles per PFU). The weight of virus shed might be greater by a factor of 10 to 100 because most other virus particles are larger than those of enteroviruses. Given a number of additional assumptions, one can predict, very approximately, the presence of 1 PFU of virus per milliliter of urban raw wastewater.

There is no reason to believe that the rate of virus infection would differ significantly among the 25 percent of the United States population that is served by private waste disposal systems. However, intestinal virus infections are likely to occur in a family on an all or none basis, (most usually none) so that average levels of virus incidence are not meaningful. The great majority of private waste disposal systems would be expected to receive no virus at all on a given day; whereas those that do receive virus are probably getting a hundred-fold higher level than that in urban sewage.

Waste Disposal--

The main line of defense against transmission of viruses shed in feces is proper waste disposal. A private waste disposal system most often comprises a septic tank and a soil absorption field. The virology of the septic tank is virtually unknown; whereas, studies on transport and inactivation of viruses in soils abound. There have been three general kinds of studies:

a. Batch studies, in which the virus is added to a slurry of sand or soil particles in some type of fluid -- The mixture is stirred for a period of time; the fluid is separated from the solids by settling, centrifugation, or filtration; the quantity of virus remaining in the fluid is measured; and the degree of virus adsorption to the solids calculated. Batch studies are not of direct applied value because the extent of virus removal is small. They enable determination of how ions, pH and interfering substances affect virus adsorption.

b. Column studies in which virus-containing fluids are passed through tubes packed with sand or soil. These simulate the filters of disposal beds being evaluated. Columns are especially useful in measuring the effect of flow rate on adsorption. Virus adsorption is orders of magnitude greater in columns than in batch tests. This is probably due to the increased opportunity for contact between virus and fill material in the columns.

c. Field studies, in which samples are taken from wells dug at known distances from a virus-application site. These studies document infectious virus movement under natural conditions. Insofar as they are meant to characterize the movement of naturally-occurring virus in wastewaters, they may be limited by the infrequent incidence of viruses in individual household wastes. Negative test results may not mean anything, and positive test results may not mean what they seem.

Virus Adsorption--

Some general principles emerge from the studies on virus adsorption to particles of soil and sand.

Batch studies--Drewry and Eliassen (1968) and Eliassen and Drewry (1965) carried out batch-type adsorption studies with three ^{32}P-labeled bacteriophages suspended in distilled water containing various concentrations of sodium and calcium salts. Five soils were tested. Virus adsorption decreased with increasing pH, in the range from 6.8 to 8.8, and increased with increasing cation concentration. Three soils showed greater than 90 percent adsorption at all concentrations. Virus adsorption also increased with increasing ion-exchange capacity, clay content, organic carbon and glycerol-retention capacity of the soil. However, one soil, which ranked low in these properties, had the greatest adsorptivity for viruses.

Carlson, et al., (1968) measured adsorption of poliovirus 1 and bacteriophage T2 on clay suspensions. Adsorption was negligible if the suspending medium was distilled water; cations enhanced adsorption as a function of their concentration. Their relative effectiveness was $Al^{3+} > Ca^{2+} > Na^{+}$. The process was 90 percent complete in 5 min and essentially 100 percent complete in 20 min. Adsorption could be prevented or reversed by adding albumin to the mixture. Adsorbed virus remained infectious.

Schaub and co-workers (1974, 1975) studied the adsorption of enteroviruses poliovirus 1, coxsackievirus B-2 and encephalomyocarditis virus to clays. With respect to cation concentration, adsorption to bentonite was maximized (97%) with 5mM Ca^{2+}; 100 times greater concentration of Na^{+} was required for equivalent adsorption. Enterovirus adsorption to montmorillonite did not vary significantly over a pH range of 3.5 to 9.5. The virus association appeared to be stable over a prolonged period, but 22 to 30 percent could be eluted readily by diluting the suspension with 99 volumes of estuary water from which solids had been removed by filtration. Inactivation of sorbed and free virus proceeded at similar rates. Sorbed virus was infective to laboratory animals.

Column studies--Robeck, et al,(1962) applied poliovirus 1 (Po-1) in 80 to 160 cm/day of dechlorinated tap water to a column filled with 60 cm of California dune sand. The column removed 2.7 to 4.8 $\log_{10}$ of virus continually for 98 days. With 60-cm columns of fine and coarse Ottawa sand, virus removal increased as flow rate decreased; there was a large variation from run to run at some of the rates, however. In the coarse (0.78 mm) sand column, virus removal varied from 1 percent at a flow rate of 7.6 cm/min to greater than 98 percent at 43 cm/day. Each run was continued long enough to displace 2.5 times the volume of liquid held in the column.

In upflow studies, columns containing 60 cm of coarse (0.38 mm) Chillicothe, or fine (0.18 mm) Newtown sand were dosed with water containing 10^4 plaque forming units (PFU)/ml and flowing up through the columns at a velocity of 90 cm/day. Complete displacement of the fluid in the column occurred in 16 to 24 h. The effluent from the fine sand column was "virtually" virus-free; the effluent from the coarse sand column averaged 8 PFU/mL. No exhaustion of virus-removal capacity was observed during seven months of continuous operation. Addition of alum to a virus suspension before filtration through coarse sand resulted in virus-free effluents for a limited time, but after 6 to 7 h both the floc and the virus broke through.

Drewry and Eliassen (1968) and Eliassen and Drewry (1965) reported studies in which ^{32}P-labeled bacteriophages T1 and T2 were applied to 43- to 50-cm soil columns. The columns were operated at 20° C under saturated, continuous downflow conditions (18 to 41 cm/day). The phage was suspended in distilled water containing 1.0 mM Ca^{++} and 1.7 mM Na^+. The columns removed from 79 to over 95 percent of the label, depending on the type of phage and soil tested. Most of the phage was adsorbed in the first few centimeters. A significant amount of inactivation occurred as a result of passage through the columns: PFU to ^{32}P count per minute ($\frac{PFU}{CPM}$) ratios were 2 to 4 $\log_{10}$ lower in the effluents than in the feed solutions.

Hori, et al., (1971); Tanimoto, et al., (1968) and Young and Burbank (1973) tested the adsorption of coliphage T4 and poliovirus 2 (Po-2) to three types of soil, packed into 3.8 to 15-cm deep columns. The soils were two clay-type, low humic latersols, and a volcanic cinder. The virus was suspended in distilled water and the columns were washed with distilled water before dosing at 15 cm/day. Virus breakthrough occurred in all the columns and the levels of virus in the effluents increased with time. The clay-type soils were similar to each other in performance, and more retentive of virus than the cinder. T4 was retained more effectively than Po-2. After 5 bed volumes of the suspension had been treated, the 15-cm clay-type columns still retained over 99 percent of the poliovirus, whereas only 22 percent was retained by the cinder column. All of the T4 was retained by the 15-cm clay-type columns, and more than 52 percent by the 15-cm cinder column.

Nestor and Costin (1971) constructed 70-cm columns with fresh sand and with sand from operating waste-treatment filters. The sand was analyzed for organic matter: the fresh sand contained 0.21 percent and the used sand 1.45 percent.

About 4.7 cm of water was used to moisten dry, fresh sand columns. The columns were dosed with 15.6 cm of tap water containing coxsackievirus A-4. Two dry (sic) fresh sand columns reduced the titer of the virus suspension by 11 percent; two moistened fresh sand columns reduced it by 80 percent; used sand from an unwashed sand filter removed 95 percent; and used sand from a washed filter removed 98 percent of the virus.

Kott and co-workers (Goldsmith, et al., 1973; Lefler and Kott, 1973, 1974) dosed 10-cm fresh sand columns with 40 cm of salt solution containing Po-1 or bacteriophage f2 with similar results. Virus retention was related to cation type and concentration. With 1.0mM NaCl, 18 percent of the poliovirus was adsorbed; with 500 mM NaCl, 83 percent of the virus was retained. Retentions were 45 and 14 percent respectively, with 0.5 mM Ca Cl_2 and $MgCl_2$. Increasing the divalent cation concentrations to 5.0 mM resulted in virus retentions of 99.98 percent, which was similar to results observed with tap water.

Duboise, et al., (1974) studied migration of Po-1 and bacteriophage T7 through relatively porous sandy soil cores 19.5 cm high and 6.3 cm in diameter. The cores had been washed with deionized water. The viruses were suspended in deionized water and applied in small bands at the tops of the columns, followed by either continuous (4-cm pond) or intermittent application of deionized water. Three cores receiving T7 under continuous flow conditions retained 99, 96 and 93 percent of the virus. There was no correlation between virus breakthrough and the quantity of fluid accepted by the columns. Virus breakthrough was rapid, decreasing gradually with time. Poliovirus behaved in a similar fashion; 98 and 99.5 percent were retained by two columns. Under intermittent flow conditions (1 cm of dionized water every 4 hours), 99 percent of the T7 and 99.6 percent of the poliovirus were retained.

The most important factor in virus retention by soil columns appears to be flow rate: rapid flow systems generally removed only a small percentage of the applied virus, while columns receiving moderate amounts of fluid reduced virus levels by several orders of magnitude. Increasing cation concentration in the suspending fluid also enhanced virus adsorption.

Field studies--In a wastewater reclamation project at Whittier Narrows, California (McMichael and McKee, 1965), wastewater containing 3×10^3 PFU of poliovirus 3 (Po-3) per liter was applied to a percolation basin. Samples were collected from a central well with sampling pans at 61 to 244 cm depths. The soil in the basin was described as "dark brown very fine to medium silty sand and soil, with an abundance of organic material." None of the samples were positive for virus; but, because concentration techniques were not used, the results could only be reported as less than one PFU per milliliter.

Wastewater containing added Po-3 was reclaimed by percolation through soil in another California study (Merrell, et al., 1967). Wells were installed in a sand and gravel formation at 61, 124 and 458 m from the edge of the disposal bed to sample the upper 15 to 30 cm of groundwater. Approximately 5.3×10^5 liters of wastewater containing 10^3 tissue culture infective doses per ml were applied to the percolation bed. No virus was detected in any of the samples taken from the wells over a 49-day period. Later tests showed that a salt solution applied to the percolation bed reached the wells in less than 48 hours.

Poliovirus 2 (Po-2) was isolated from drinking water from a well located 92 m from the edge of a septic tank drainfield (Mack, et al., (1972); Vander Velde, 1973). A geological survey of the area revealed that the soil underlying the absorption field was shallow and clayey, so there was probably little treatment of the wastewater before it reached the groundwater and traveled rapidly through cavernous limestone to the well.

Wellings, et al., (1974) sampled water from wells draining a spray irrigation site which consisted of about 4 ha (10 acres) of sandy soil irrigated with effluent from an activated-sludge waste-treatment plant. Application rates ranged from 5 to 28 cm per week. Virus traveled up to 6 m in the sandy soil. Most of the isolations were made following heavy rains.

Wellings, et al., (1975) also investigated the virological aspects of a scheme to discharge treated wastewater into cypress domes, which are continuously ponded. The dome under study had been drained, however. The soil profile showed alternating clay and sand layers; the major groundwater flow occurred in the sand layers. Sampling was by means of eleven drilled wells 3 m deep, lined with polyvinyl chloride pipe. Most of the samples were negative, but a grand total of 3 PFU were detected in the course of the entire study. The isolations demonstrated percolation of virus to 3 m depth, plus lateral flow of at least 7 m. Virus passage through the clay could have been aided by the construction of an observation tower, the support pilings of which cut through the clay layers. Two of these 3 PFU were detected 28 days after the last effluent was applied, indicating virus persistence of at least that long in the dome. These isolations followed heavy rains, which might have aided in the transport of the virus.

Dugan, et al., (1975) analyzed leachates from 150 cm deep lysimeters dosed with wastewater. The lysimeters were filled with various types of soil. One sample out of 28 was positive for virus. When "high concentrations" of Po-1 were added to the wastewater, virus was found to penetrate as deep as 117 cm in one case (sod lysimeter), and 15 cm in another (bare soil lysimeter). Another bare soil lysimeter was seeded in two ways: either the virus was diluted in a large volume of wastewater or the concentrated virus was applied directly to the soil. In the first case, virus was detected as deep as 10 cm; in the second, the virus had penetrated to 91 cm depth.

These field studies, although mostly uncontrolled, indicate that while soil disposal beds are generally effective in preventing virus entry into groundwater, heavy rains or breaks in the soil strata might reduce the effectiveness of the system.

Summary--In terms of the objectives of the Small Scale Waste Management Project, a number of important questions remain unanswered by the investigations reported in this section. A few of the studies used wastewater as the suspending medium for the virus, but little attention has been paid to the changes which might occur in the system after many months of wastewater application. These changes could alter the ability of a soil to remove viruses. Secondly, it is not possible to predict, on the basis of the information available, just how much virus would be removed by a given system. Although flow rates seem to influence virus penetration into soil, the available data do not allow selection of a rate which would ensure complete safety. There is no information on the effect of low temperatures on virus retention, and the question of whether or not the virus is inactivated after sorption has not been addressed. Recent reviews pertinent to this subject have been written by Bitton (1975) and by Gerba, et al., (1975).

Laboratory and Field Studies

Areas of Study--

Several aspects of the virology of on-site disposal systems have been studied. The virology of the septic tank and the virology of septage were examined briefly. The principal focus has been on the removal of virus from septic tank effluent (STE) by slow percolation through 60 cm of sand, as in a mound. The virology of systems which might be used to prepare STE for surface disposal was also investigated.

Materials and Methods--

Virus preparation and assay--Poliovirus type 1 (Po-1), strain CHAT, was obtained from the Viral and Rickettsial Registry of the American Type Culture Collection. Po-2 and Po-3 were isolated from feces of a recently vaccinated child. A swine enterovirus, ECPO-6 was obtained from Dr. E. H. Bohl at the Ohio Agricultural Research and Development Station, Wooster.

The polioviruses were propagated in primary rhesus (*Macaca mulatta*) monkey kidney (PMK), established African green (*Cercopithecus aethiops*) monkey kidney, or established human cervical carcinoma (HELA) cell cultures. An established swine kidney cell culture lines (MPK) was used to detect the ECPO-6 virus, which served as a model contaminant in the feces of an infected pig. Methods for preparing the cultures, propagating the viruses, detecting viruses in cultures maintained with fluid medium, and quantification of viruses by the plaque technic in cultures maintained with semisolid medium were largely as described earlier (Cliver and Herrmann, 1969; Salo and Cliver, 1976).

Po-1 was sometimes labelled with ^{32}P so that the virus particles could still be located, even if they were no longer infectious. The procedures used have been described in detail elsewhere (Cliver and Herrmann, 1972; Herrmann and Cliver, 1973; Salo and Cliver, 1976).

Sample processing--The samples to be examined in a study such as this differ considerably from those ordinarily encountered in laboratory virology. Special procedures had to be developed to make the samples suitable for the detection and assay of virus that was present.

Samples derived from an experimental septic tank at the Park Street Laboratory (site N) presented unusual contamination problems. Trial and error led at last to a procedure for coping with this contamination. To a 10 mL sample in a large screw-cap tube were added 0.05 mL diethyl ether. The mixture was treated for 15 sec with the tube immersed in ice water in a sonic cleaning bath. After 2 hours of inculation at room temperature, air was sparged through the sample for 1 hour to remove the ether residue. This reduced the microbial population but did not eliminate it. When the sample was tested in tissue cultures, it was necessary to include the following in each milliliter of maintenance medium:

5 µg amphotericin B, 1500 µ penicillin, 60 µg tetracycline, and 100 µg gentamicin.

Samples of septic tank effluent (STE) from an operating septic tank at the Arlington Experimental Farms and of fluids partly or completely treated in a sand or soil column were processed somewhat differently. If such samples were frozen before testing, a precipitate would often form; when this precipitation occurred, most of the virus that had been present was lost in some unexplained way. Therefore, samples were ordinarily stored in the liquid state, at temperatures $\leq 8^{\circ}$ C. If the level of virus in the sample was expected to be fairly high, the sample might be diluted with 9 volumes of phosphate-buffered saline PBS) plus 2 percent fetal calf serum (PBS-FC_2) and then frozen. Either of two means were used to rid these samples of bacteria before testing in tissue culture: 1) adding chloroform (5% by volume) to the sample, mixing and then removing all traces of chloroform by sparging air through the sample, or 2) filtration at 0.20 µm porosity (Gelman GA-8) (Cliver, 1968). The latter method required that suspended solids be removed from the sample by prefiltration. Effective prefilters tended to adsorb virus. This could be prevented by adding 2 percent fetal calf serum to the sample before prefiltration (Cliver, 1965); however, the presence of the serum would preclude later concentration of virus by the adsorption-elution method (Cliver, 1967). If the virus had adsorbed to the solids which were to be removed by the prefilter, the adsorption could usually be reversed by adding 10 percent fetal calf serum. Virus could be concentrated from a sample by adsorption to a cellulose nitrate membrane filter (0.22 µm porosity, Millipore GS or 0.45 µm porosity, Millipore HA and elution with a small volume of fetal calf serum or cold pH 11.5 buffer. However, adsorption was likely to be poor if calf serum had been added to the sample to enable prefiltration, and elution was often less than complete.

Sorbed virus was removed from sand and soil column fill samples by stirring with 1.0 mL of fetal calf serum per gram of fill. This procedure was tested by adding 5.0 mL of ^{32}P-labeled Po-1 to 22.6 g of sand, conditioned as described below by use in STE treatment, in a chromatography column. The virus was allowed to adsorb overnight, eluted as above, and the eluate counted. Recovery was 77 percent. Recovery was not improved by adjusting the pH of the serum to 8, 9, 10, 11 or 12. Recovery with pH 11.5 glycine buffer was 71 percent; with PBS as the eluent, recovery was 20 percent.

Yet another procedure was used to recover virus from septic tank sludge with relatively little admixture of supernatant after experimental inoculation and attempts at disinfection. An equal volume of fetal calf serum was added to the sample, and the mixture was stirred with a magnetically-driven bar while the pH was adjusted to 9 with 1N NaOH. After another 15 minutes of stirring, the mixture was treated with 5 percent chloroform by volume for 1 hour in an ice bath. Settled solids were resuspended and the suspension was decanted from the chloroform. Residual chloroform in the suspension was removed during 15 minutes of treatment in a sonic cleaning bath filled with ice water; the flask containing the sample was subjected to a vacuum of 500 mm of mercury to encourage vaporization of the chloroform. The treated sample was stirred for 15 minutes with an equal volume of phosphate-buffered saline (PBS) and assayed by the plaque technique. Using ^{32}P-labelled Po-1, this procedure was shown to recover all of the virus particles, but more than 40 percent of the infectivity was lost in the process. Sodium sulfite (Na_2SO_3) was used to neutralize glutaraldehyde in samples from the disinfection experiments. The final concentration of 1.26 g/L (10mM) was shown not to affect the infectivity of Po-1.

Columns--The majority of experiments were so varied that the procedures can best be described together with their results. However, the operation of the various sand and soil columns used in treating STE had enough features in common to merit a general description.

The first columns (A and B) were the largest. They were constructed from 14.6 cm ID PVC pipe, 150 cm in height and were capped at the bottom end (Figure C-8). The cap was fitted with an outlet tube. The columns were filled with 15 cm of gravel at the bottom; 30 cm of a Batavia silt loam soil; 60 cm of a medium (0.2 mm) sand from the C horizon of a Plainfield loamy sand; and 30 cm of gravel (Magdoff, et al., 1974a).

Below the sand surface were rings of 1 cm diameter aeration holes spaced 8 cm apart. Also below the sand surface were 2.4-cm holes (plugged with rubber stoppers) for fill sampling. Samples were obtained by removing a stopper and digging out some fill with a bent-tipped spatula. For fluid sampling, fritted-glass tipped gas-dispersion tubes were embedded in the fill at the same depths as the fill sampling ports; fluid samples were drawn by applying a vacuum.

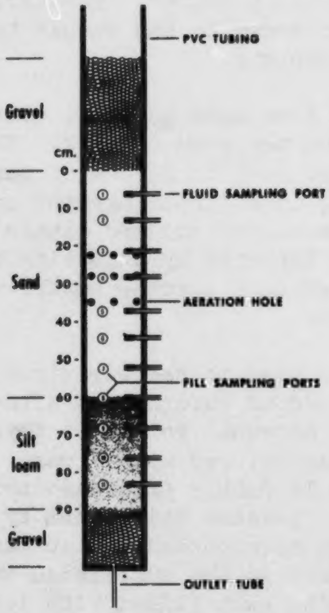

Figure C-8. Diagram of Column A (14.6 cm inside diameter)

All columns were maintained at room temperatures (18° to $24^{\circ}C$), unless otherwise stated, and were dosed manually once a day with STE from a family residence. The STE was delivered to the laboratory weekly in a 19 L glass carboy. This volume was divided into dose-size units and stored at 6° to 8° C. Units were warmed to room temperature, and virus was sometimes added just before dosing the columns. Columns A and B initially absorbed approximately 3.5 L of STE before any effluent began to flow out the bottom port.

The 60 cm of sand in the 14.6 cm ID columns occupied a volume of 10,000 cu cm and weighed 17.2 kg when dry. When a fill sample was saturated with water, its weight increased to 121 percent of its dry weight, so the void volume of a 60 cm, 14.6 cm ID sand column was approximately 3.6 L. In normal operation, the columns were never saturated. Fill samples taken 24 h after dosing with 5 cm of STE showed that the sand at the top of the column was about 25 percent saturated, increasing to 100 percent at 60 cm. Calculations indicated that a 5 cm dose resided in the sand for two full days.

Columns C, D, E and F were 7.6 cm ID and were filled with 60 cm of sand, without any underlying silt loam. Columns C, D and F were filled with fresh sand, whereas column E (also containing 60 cm of sand) had been dosed with 5 cm of STE/day since its construction 3 weeks earlier.

Two columns contained Almena silt loam cores which had been collected *in situ*. The cores were 48 cm (column G) and 54 cm (column H) high and were set in 66 cm sections of 10 cm ID acrylic pipe. They received 0.6 cm (50 mL) of STE daily, the maximum they would accept being 65 mL. NaCl (300 ppm) was added to the STE to determine its rate of passage through the columns; the results indicated that column G may have been short-circuiting.

Shorter, smaller columns (minicolumns) were sometimes used in order to get a clearer picture of the movement and inactivation of viruses in sand columns. Cylindrical filter holders (Millipore Corp. or Gelman Co.) were found to be ideal for this purpose. The Millipore holders consisted of a 1.6 cm ID glass cylinder which clamped to a glass base containing a stainless steel mesh filter support. The Gelman units were composed of 1.9 cm ID polypropylene tube which attached to a stainless steel base fitted with a steel mesh. For the Millipore columns each milliliter of dose volume was equivalent to 0.50 cm height; in the Gelman units a 1.0 mL dose was equivalent to 0.35 cm.

The units were assembled without a filter, and packed with 1 to 5 cm of sand or soil, as required. The approximate void volume of the minicolumns was determined by dosing a moistened and drained 3 cm fresh sand column with 0.5 cm (1.0 mL) of PBS labelled with $^{36}Cl^{-}$. This was followed by 0.5 cm doses of label-free PBS. The first 0.5 cm of effluent contained none of the label; the second, 3 percent; the third, 55 percent; diminishing thereafter. From these data the void volume was calculated to be 0.3 cm of fluid per cm of fresh sand. The experiment was also performed using conditioned sand, which showed a similar void volume.

Results--

Septic tank inoculation--Infectious swine feces (25g) containing 10^7 PFU of ECPO-6 were added to the input end of the 1000 gallon septic tank at the Park Street laboratory (site N). If the tank had contained exactly 1000 gallons of liquid and if mixing had been instantaneous and complete, each milliliter of the contents should have contained ~ 65 PFU of the virus. None were detected in effluent samples taken on 18 successive days thereafter. No firm conclusions can be drawn from this because the contamination problems described previously had not been solved fully by the time of this experiment. However, it does appear that this septic tank may have inactivated much of the virus with which it was inoculated.

Column studies--The purposes of these studies were to determine the degree to which virus in STE was removed by passage through sand and other soils, the effect of prolonged use upon sand and soil performance, the influence of the STE loading rate, the influence of temperature and the extent to which the retained virus is inactivated (loses infectivity).

The sand or soil into which STE is discharged starts in a "fresh" state and gradually becomes "conditioned with use." The study first tested retention of virus by these media when they were in the "fresh" state. Two 5.0 cm ID columns were filled with 10 cm of fresh sand, and two with 10 cm of fresh Batavia silt loam soil. Each was dosed daily with 5 cm (100 mL) of STE containing 12,000 PFU of Po-1/mL. Effluents were collected daily and assayed for virus infectivity (Table C-7). The sand column effluents contained fewer than 7 PFU/mL for three days; then the titers increased by an order of magnitude. This was perhaps a result of the columns becoming conditioned. The daily dose exceeded the functional void volume of the sand. Virus remained low or undetectable in the effluents from the Batavia silt loam columns.

Further, to establish the degree of virus retention by fresh soils, two 1 cm minicolumns were constructed. They were packed with Almena and Batavia silt loams respectively. Each was dosed with 1.8 cm of Whatman #1 filtered STE containing 8×10^7 PFU of Po-1/mL. The effluent from the Almena column contained 1 PFU/mL, a 7.9 log reduction; the Batavia column effluent contained 1.7×10^2 PFU/mL, a 5.7 log reduction.

The effect of conditioning--Penetration of the virus into the fresh sand columns progressed with time. This could have been the result of repeated desorption, transport and readsorption with each STE dose. Another possibility was that as the columns became conditioned, their retentiveness for virus decreased.

To test the effect of conditioning on virus retention, the performance of 60 cm of fresh sand (column F) was compared with 60 cm of sand (column E) which had received 5 cm of "virus-free" STE daily for 3 weeks. Column F was given a single virus-free STE dose, and both columns were placed in the cold room (6° to 8° C) to retard conditioning in column F. The next morning (day 0) both columns were dosed with 5 cm (250 mL) of (≤ 8° C) STE containing 6.0×10^8 PFU of Po-1/mL. Thereafter, each column received daily 5 cm doses of cold (≤ 8° C) virus-free STE. Titers of fluid samples taken from various depths in these

TABLE C-7. REMOVAL OF Po-1 FROM STE BY 10 cm FRESH SAND AND SOIL COLUMNS[1]

Day	Effluent titers in PFU/mL from Sand Columns		Soil Columns	
	#1	#2	#1	#2
0	0[2]	0	0	0
1	0.3	0	0.2	1.5
2	1	2	0	0
3	7	2	0	0
4	19	32	0	6
5	54	20	0	0
6	50	32	0	1.7
7	36	12	0	0
8	33	0[2]	0[2]	1
9	35	0	0	2
10	51	1	0	0

[1] Daily dose: 5 cm of STE containing 12,000 PFU/ml.

[2] No PFU detected = 0.

[3] Column ponded.

columns are reported in Table C-8. The conditioned column was less retentive of virus than the fresh sand column.

To examine further the difference in virus adsorption between fresh and conditioned sand, two sets of minicolumns were constructed. Each set consisted of three columns in Gelman filter holders, packed with 2, 4 or 6 cm of fill. One set contained fresh sand, the other conditioned sand removed from columns A and B. Each column was dosed with 3.5 cm of labeled virus. The effluents were collected and counted. Results are presented in Table C-9. The fresh sand was 1.8 times more effective in removing virus than the conditioned sand, and the removal was proportional to column length.

TABLE C-8. VIRUS TITERS OF FLUIDS FROM COLD SOIL COLUMNS[1]

Day	Column E (Conditioned) (cm depth)				Column F (Unconditioned) (cm depth)			
	5	15	30	45	5	20	30	45
1	6[2]	6	1.3	0[3]	5.04	0.5	--	--
2	5.54	5.84	3.04	0	3.95	0.95	--	--
3	5.36	5.23	2.8	0.2	3.77	0.2	0	0
5	--	--	--	1.57	4.56	0.8	--	--
8	--	5.00	2.5	0.3	--	0	0	--
14	5.28	5.59	2.8	0.6	5.00	0.8	0.0	--
21	5.32	5.18	3.65	0.2	5.08	1.3	1.76	0
35	4.95	5.42	3.64	0.0	5.00	1.43	0	0
175	2.90	3.36	3.01	1.53	3.63	2.93	1.58	0
180	4.79	4.74	3.53	0.6	4.99	2.5	0.0	0
205	3.93	3.23	2.45	0.0	3.76	2.00	1.00	0
362	2.3	2.9	2.58	0	3.51	2.88	0	0
363[4]	2.3	2.8	2.0	0	3.0	1.8	0	0

[1] Dosed daily with 5 cm of STE, which contained 8.78 $\log_{10}$ PFU of Po-1 per mL on day 0 and 7.00 $\log_{10}$ PFU of Po-2 per mL on day 176. On day 175 they received 20 cm of virus-free STE. They were maintained at 6-8° C, but column E had been preconditioned at room temperature.

[2] $\log_{10}$ (PFU/mL)

[3] No virus detected = 0

[4] The viruses in these samples were mostly Po-1.

Two minicolumns were filled to 1 cm with fresh Almena silt loam, and two with fresh Batavia silt loam. The soils were conditioned by dosing the columns for 28 days with 0.35 cm of filtered STE/day. On the 29th day, 0.9 cm of filtered STE containing 9.0×10^7 PFU was added, followed by daily 0.35 cm doses of virus-free filtered STE. Accumulated effluents were collected 1 and 9 days after the

TABLE C-9. COMPARISON OF Po-1 RETENTION BY FRESH AND CONDITIONED SAND COLUMNS

Fill	Column Length (cm)	Recovery in Effluent (%)	Loss/cm ($\log_{10}$)
Fresh	2.0	15	0.41
	4.0	2	0.43
	6.0	0.5	0.39
Conditioned	2.0	50	0.15
	4.0	22	0.16
	6.0	11	0.15

the virus dose. The day 1 effluents from the Batavia columns contained totals of 270 and 300 PFU; by day 9, only another 140 PFU had washed out of the first column and 22 PFU from the second. Approximately 5.4 $\log_{10}$ of virus were retained by the columns. No virus was detected in the effluents from the Almena columns, showing at least a 7.9 $\log_{10}$ reduction in virus titer. These values correspond quite closely to the results obtained with the fresh soils, there was no indication that conditioning had reduced their retentiveness for virus.

Effect of hydraulic loading--Fresh 10-cm sand columns had removed 4 $\log_{10}$ of virus from a daily 5 cm dose of STE. The next problem examined was the effect of a 10-fold increase in hydraulic loading rate. At higher loading rates the pores between the sand grains would be filled with fluid, and this could affect virus adsorption. Two 60 cm fresh sand columns were dosed daily with STE containing ca. 10^6 PFU of Po-1/mL. Column C received 5 cm/day and column D 50 cm/day. Virus concentrations in the columns are represented in Figure C-9. At 10 cm depth on the first day (day 0), the 5 cm dose was reduced in titer by 4.4 $\log_{10}$; the 50 cm dose was reduced by only 3.2 $\log_{10}$. As time progressed, virus penetrated deeper and in greater numbers in both columns, but column D samples were always one or more logs higher in titer than the comparable sample from column C (Tables C-10 and C-11).

An experiment to determine the effect of an STE overload on a conditioned 60 cm column, was carried out on column A which had 30 cm of Batavia silt loam below the sand zone (Figure C-8). An 8.0 L batch of STE was inoculated with Po-1 to a titer of 4.6×10^5 PFU/mL. This amount of STE was equivalent to a 50 cm dose, 10 times the "normal" amount. One-tenth of this was applied to the column, and as soon as no fluid remained on the surface, fluid samples were taken from ports 1 through 8 and 12. An additional 40 percent was then applied and was followed by an identical sampling procedure. The final 50 percent was then applied; it percolated through the column during the night. The next morning, before dosing, a third set of samples was collected. A fourth set

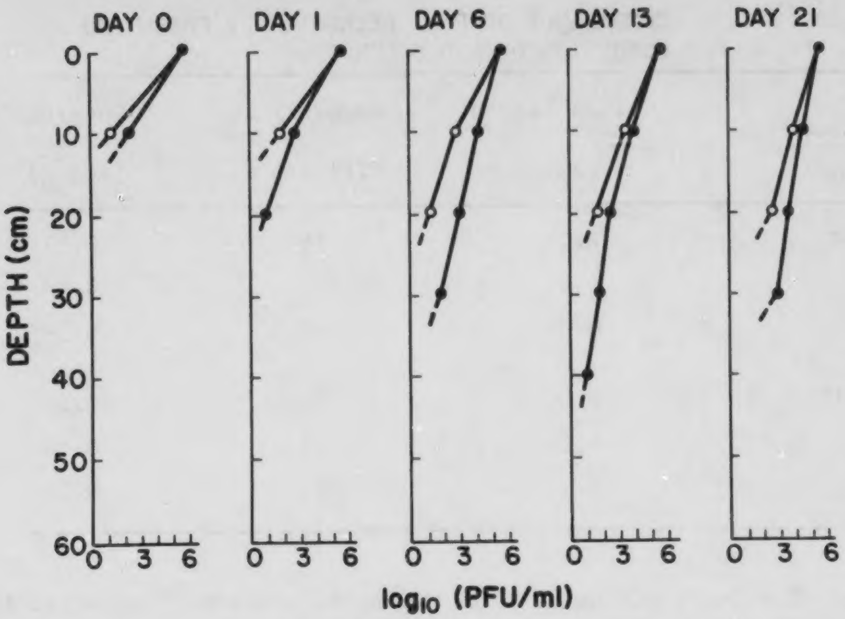

Figure C-9. The effect of dose size on Po-1 penetration into 60 cm fresh sand columns. Symbols: 0, 5 cm of STE/day; 50 cm of STE/ day until day 11, when crusting reduced the flow rate to ca. 20 cm/day.

was collected five days later, after normal dosing (5 cm of virus-containing STE/day) had resumed. The data showed that as the dose increased, the virus penetrated further and in greater numbers through the column. By the second sampling (5 cm + 20 cm STE applied) the virus had penetrated as far as the bottom of the sand; and by the third sampling (5 cm + 20 cm + 25 cm = 50 cm STE applied) virus was detected in the effluent, having passed through the silt loam in the bottom of the column as well. After returning to normal dosing, the virus levels dropped at all depths, but still had not returned to pre 50 cm dose levels after 5 days (Table C-12).

Next, the effect of flow-rate on virus retention by conditioned sand was studied. Two minicolumns were filled to 3 cm with sand from columns A and B. Each was dosed with 3.5 cm (10 mL) of PBS containing 5,400 CPM of ^{32}P-labeled Po-1/mL. One of the columns had the entire dose applied directly: This passed through the column in less than one minute. The dose for the other column was applied dropwise over a 2.5 h period (1.4 cm/h). The rapid flow column retained 61 percent of the virus; whereas the slow-flow column retained 96 percent. The slow-flow column was 2.2 times as retentive per centimeter of column length as the rapid-flow column, in terms of log$_{10}$ virus titer reduction.

TABLE C-10. VIRUS TITERS OF FLUID SAMPLES FROM COLUMN C

		Sampling depth					
Day	STE	10 cm	20 cm	30 cm	40 cm	50 cm	60 cm
0	5.6[1]	1.23	0[2]	0	--	--	--
1	--	1.8	0	0	0	--	0
6	--	2.81	1.18	0	0	--	--
13	--	3.5	1.60	0	0	--	--
20	--	--	--	0	--	--	0
21	6.15	4.00	2.6	0	0	--	0
22a[3]	6.00	4.80	3.15	1.40	0	0	0
22b[4]	--	4.81	3.58	2.64	0.8	0	--
23	--	4.58	3.8	2.0	0.3	0	0
26	--	5.08	4.63	3.32	2.64	0	--
30	--	5.68	4.54	3.18	3.08	0.8	0
57	5.48	5.65	5.3	3.6	0.7	0	0
70	6.45	6.34	6.34	--	--	0.7	0
84	--	5.45	4.78	3.49	1	1.11	1.08
92	5.93	--	--	--	5.84	1.04	0
99	5.85	5.26	5.11	4.8	4	--	2.8
107	--	--	--	--	--	--	2.6
114	--	--	--	--	--	--	1.95
120	--	--	--	--	--	--	1.8
137	--	--	--	--	--	--	1.70
142	--	--	--	--	--	--	2.63
147	--	--	--	--	--	--	0.3
154	--	--	--	--	--	--	0
160	--	--	--	--	--	--	0.6
175	--	--	--	--	--	--	0
182	--	--	--	--	--	--	1.70
191	5.2	--	--	--	--	--	--
200	--	--	--	--	--	--	0
208	--	--	--	--	--	--	0
217	--	--	--	--	--	--	0

---

[1] $\log_{10}$ (PFU/mL)

[2] No virus detected = 0

[3] 5 cm dose

[4] 50 cm dose

TABLE C-11. VIRUS TITERS OF FLUID SAMPLES FROM COLUMN D

		Sampling depth					
Day	STE	10 cm	20 cm	30 cm	40 cm	50 cm	60 cm
0	5.6[1]	2.38	0[2]	0.6	0	0	0
1	--	2.58	0.8	0	0	--	--
6	--	4.15	2.95	1.80	0	--	--
13	--	3.8	2.5	1.81	0.95	--	--
21	5.0	4.53	3.69	2.95	0	0	0
26	--	4.18	3.8	2.5	2.67	0	0
30	--	5.02	4.63	3.34	2.84	0	0
57	5.48	5.64	5.71	5.69	5.36	2.11	2.48
70	6.45	6.28	5.65	5.26	--	4.00	4.08
84	--	5.54	5.46	5.30	5.20	4.62	3.90
92	5.93	--	--	--	4.7	4.5	4.3
99	5.85	5.67	5.47	4.60	4	4	3.1
114	--	--	--	--	--	--	3.1
120	--	--	--	--	--	--	2.8
136	--	--	--	--	--	--	1.3
137	--	--	--	--	--	--	1.9
142	--	--	--	--	--	--	1.28
147	--	--	--	--	--	--	0.3
154	--	--	--	--	--	--	0
160	--	--	--	--	--	--	0.7
175	--	--	--	--	--	--	0.8
191	5.2	3.5	1.7	0	1.5	0.7	0.5
200	--	--	--	--	--	--	--
208	--	--	--	--	--	--	0
217	--	--	--	--	--	--	0

[1] $\log_{10}$ (PFU/mL)

[2] 0, no virus detected.

If virus retention is a function of flow-rate, then virus adsorption may be related to the degree of fluid saturation of the pores between the sand grains. To investigate this, a minicolumn filled with 2 cm of fresh sand was washed with PBS and drained; the last free liquid was forced out with compressed air. Labeled virus in 0.35 cm (0.6 times the void volume of the sand in this minicolumn) of phosphate buffer was added, followed immediately by 3.2 cm (5 times the void volume) of PBS. Approximately 98 percent of the virus label was retained in the column. The then saturated column was again dosed and flushed in the same manner. Ninety-six percent of the virus was retained. Next, 3.5 cm (6 void volumes) of the virus in PBS was added. The column retained 93 percent of the labelled virus. A second 3.5 cm dose of PBS (without virus) eluted only 0.2 percent of the sorbed virus. These data suggest that even though fresh sand retains virus quite effectively, adsorption per unit depth of sand is somewhat reduced under conditions of saturated or rapid flow.

TABLE C-12. VIRUS TITERS OF FLUID SAMPLES FROM COLUMN A, TAKEN AFTER INCREASING STE DOSE VOLUMES

Depth (cm)	STE dose (cm)			
	Cumulative in one day			
	5	25	50	5[1]
0	5.66[2]	5.66	5.66	5.30
6	4.26	5.40	--	5.04
13	3.26	4.95	--	4.6
21	2.23	3.96	--	3.3
28	2.08	3.45	4.15	--
37	-0.30	2.46	3.59	--
44	0[3]	1.34	3.28	0.0
52	0	0.70	3.00	0
60	0	0.84	2.72	1.20
90	0	--	0.30	0

[1] Five days after return to 5 cm/day dose.

[2] $\log_{10}$ (PFU/mL)

[3] No PFU detected = 0

Removal of virus from STE by rested conditioned sand--Columns C and D had "crusted" (were unable to pass 50 cm of STE per day) after 4 weeks of operation. To determine if the columns could be restored to their original state by letting them rest, dosing was stopped. Four weeks later (day 57) the columns were cooled to 6° to 8° C, and dosing with virus-containing STE resumed at the original rates (See Figure C-10).

Virus was detected in the effluent from column D immediately (See Table C-11) and in the effluent from column C after 27 days (See Table C-10). After 22 days (day 79 of total), column D was moved to room temperature, and 27 days later (day 106 of total) its dose was reduced electively to 5 cm/day. (It had not crusted.) The effluents from both columns continued to contain low levels of virus until days 125 (182 of total) and 134 (191 of total), respectively, after this experiment began.

To examine different ways in which resting could have modified column performance, barrels from 3 mL plastic syringes (Becton-Dickinson) were filled with

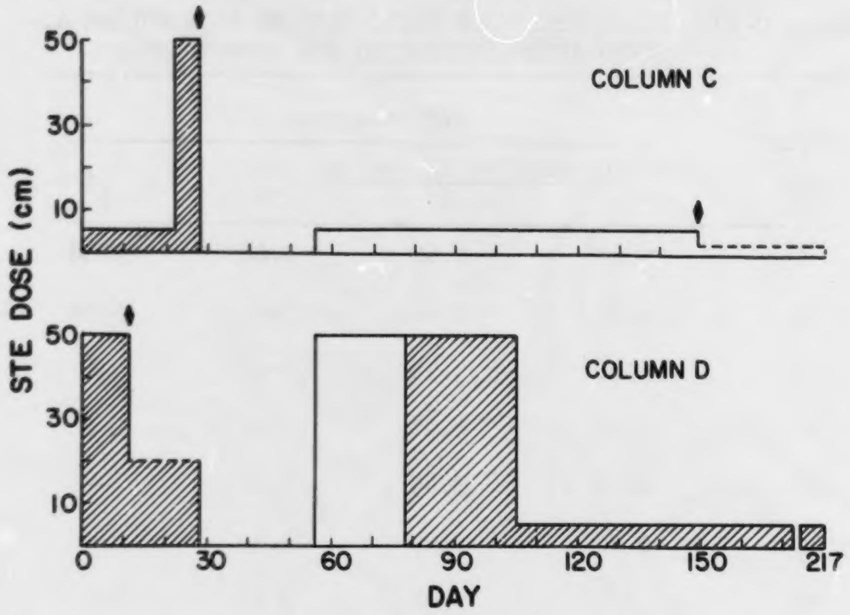

Figure C-10. Dosing and temperature regimes for Columns C and D. Shading under curve operation at room temperature; open, operation at 6-8° C; ◇ , column would no longer accept previous dose within 24 h; ----, mean dose accepted per day during period when column was ponded.

3.5 cm (2.0 cm^3) of: fresh sand; conditioned sand; air-dried conditioned sand; and acid-washed, conditioned sand. There were two replicates of each treatment. The conditioned sand columns were dosed with 0.5 mL of filtered STE for 7 days to restore the conditioning before use. Then 3.5 cm (2.0 mL or 3.3 void volumes) of ^{32}P-labeled Po-1 in GA-8 filtered column effluent was added. The first milliliter of effluent was discarded; the second was collected and assayed for label and infectivity. The columns were then dosed with 3.5 cm or filtered column effluent containing ^{32}P-labeled E. coli (Roberts, et al., 1957). The columns were all more retentive of virus than of bacteria (Table C-13), indicating that there is an adsorption rather than a filtration mechanism for virus retention. The dried, conditioned columns were less retentive of virus than the others, by two orders of magnitude. This suggest that the floc accumulated in the sand during conditioning does not adsorb virus efficiently after having been dried, and might also prevent the virus from reaching adsorptive sites on the sand surfaces.

TABLE C-13. RETENTION OF VIRUS AND E. COLI IN 3.5 cm SAND COLUMNS UNDER DIFFERENT FILL CONDITIONS

Fill	Recovery in effluent (%)	
	Po-1	*E. coli*
Fresh	0.02	11
	0.47	23
Conditioned	0.26	64
	0.30	63
Cleaned conditioned	0.20	58
	0.19	57
Dried conditioned	30	68
	46	67

Long-term operation of columns--Columns A and B contained 60 cm of sand and over 30 cm of of silt-loam soil. The columns received 8 cm (1340 mL) of STE daily for 25 days, to "condition" them. They then (day 0) were dosed with 8 cm of STE containing 3.0 x 10^7 PFU of Po-1, followed by daily 8 cm doses of STE without virus. Each day's effluent was assayed, as was the uninoculated STE. No virus was detected. Beginning two weeks after the Po-1 dose, 4.2 x 10^5 PFU of Po-3 were added each day for a week, followed by daily 3.4 x 10^6 PFU doses of Po-2. On day 28 the virus was switched to Po-1 again, at levels greater than 10^7 PFU/day. No virus had been detected in the effluents up until this time (2 mL/day were tested), but on day 28 there was a virus PFU in the effluent sample from column B. No further PFU were detected from this column. Beginning on day 37, concentration techniques were incorporated to increase the amount of effluent that could be assayed. One hundred mL or more were tested weekly, thereafter. A sample from column A collected on day 92 produced one plaque (900 mL were tested). Effluent assays were performed at intervals until day 536. From day 476, column A samples were collected from port 8, at the sand/silt-loam interface. No further PFU were detected. By day 536 the columns had each received 5 x 10^{10} PFU of poliovirus (Table C-14). A total of 23 liters of effluent had been examined for virus, and 2 PFU; one from each column were detected (Table C-15).

Effects of low temperature--Columns E and F, containing 60 cm of sand, were used to study this topic. Column E had been dosed with STE for 3 weeks at room temperature before being placed in a cold (6 to 8° C) room; the sand in in column F was fresh. Both were then dosed with 5 cm (250 mL) of STE containing 1.5 x 10^{11} PFU of Po-1; thereafter they received 5 cm/day of virus free STE.

TABLE C-14. VIRUS APPLIED TO COLUMNS A AND B

Day	Virus	Daily Dose			Cumulative dose
		Titer ($\log_{10}$ PFU/mL)	Volume (mL)	Total ($\log_{10}$ PFU)	($\log_{10}$ PFU)
0	Po-1	4.36	1340	7.48	7.48
1 - 13	None	--	"	--	7.48
14 - 20	Po-3	2.51	"	5.62	7.52
21 - 27	Po-2	3.42	"	6.53	7.76
28 - 59	Po-1	3.60	"	6.72	8.36
60 - 76	"	4.02	"	7.15	8.65
77 - 80	"	4.45	"	7.57	8.78
81 - 153	"	4.15	"	7.26	9.30
154 - 223	"	4.32	"	7.45	9.59
224 - 295	"	4.23	"	7.34	9.74
296 - 299	"	5.36	"	8.48	9.83
300 - 385[1]	"	5.57	840	8.48	10.46
386	"	5.66	8400[2]	9.56	10.51
387 - 575	"	5.30	840	8.20	10.79
576 - 611	None	--	"	--	10.79
612	Po-1[2]	8.24	"	11.17	11.32[3]

[1] Columns dosed six days per week after day 300.

[2] Column A only.

[3] Column B = 10.60

TABLE C-15. VIRUS ASSAY OF EFFLUENT FROM COLUMNS A AND B

Day	Volume assayed (mL)		Method	Cumulative volume assayed (mL)		PFU detected	
	A	B		A	B	A	B
1-27	2	2	Direct	54	54	0	0
28	2	2	"	56	56	0	1
29	2	2	"	58	58	0	0
31	4	4	"	62	62	0	0
37	400	400	3% beef extract	462	462	0	0
38	200	200	"	662	662	0	0
39	100	100	"	762	762	0	0
40	200	200	"	962	962	0	0
43	0	100	"	962	1062	-	0
44	100	100	"	1062	1162	0	0
45	200	200	"	1262	1362	0	0
52	200	200	"	1462	1562	0	0
60	200	200	"	1662	1762	0	0
73	200	200	"	1862	1962	0	0
77	200	200	"	2062	2162	0	0
87	200	200	"	2262	2362	0	0
92	900	200	"	3162	2562	1	0
103	200	200	"	3362	2762	0	0
108	200	200	"	3562	2962	0	0
110	200	200	"	3762	3162	0	0
115	200	200	"	3962	3362	0	0
128	100	100	"	4062	3462	0	0

(continued)

TABLE C-15 (continued)

Day	Volume assayed (mL)		Method	Cumulative volume assayed (mL)		PFU detected	
	A	B		A	B	A	B
133	100	100	3% beef extract	4162	3562	0	0
156	200	200	"	4362	3762	0	0
165	200	200	"	4562	3962	0	0
172	200	200	"	4762	4162	0	0
193	500	500	"	5262	4662	0	0
220	500	500	pH 11.5 glycine	5762	5162	0	0
228	450	500	"	6212	5662	0	0
240	600	600	"	6812	6262	0	0
244	650	600	"	7462	6892	0	0
261	550	550	"	8012	7412	0	0
300	380	320	Fetal calf serum	8392	7732	0	0
313	900	1150	"	9292	8882	0	0
325	340	340	"	9632	9222	0	0
328	340	400	"	9972	9622	0	0
342	320	320	"	10292	9942	0	0
393	280	340	"	10572	10282	0	0
476	320[1]	260	"	10892	10542	0	0
481	320[1]	200	"	11212	10742	0	0
519	2[1]	2	Direct	11214	10744	0	0
536	1[1]	1	"	11215	10745	0	0

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[1] Sample collected from sand/silt loam interface.

Fluid was collected periodically from the sample ports and assayed for virus infectivity.

Virus penetration was greater in the conditioned than in the fresh column (Table C-8); this pattern persisted throughout the experiment. On day 175 the columns were dosed with 20 cm (1.0 L) of virus-free STE. The next day they received 2.5 x 10^9 PFU of Po-2 in 5 cm of STE. Application of 5 cm/day of virus-free STE continued thereafter. The final samples were taken on day 363. Most of the virus in these samples was Po-1, which had been applied on day 0. Virus is less rapidly removed (and, as shown below, more slowly inactivated) in sand at 6-8° C than at room temperature. This seems unlikely to present a problem except in cases of hydraulic overloading.

Virus mobility in conditioned sand--Column A was dosed with 5 cm (840 mL) of STE containing a higher titer of virus (1.7 x 10^8 PFU/mL), followed by daily 5 cm doses of virus-free STE. The passage of the virus down the column was monitored for 24 days by assay of both fill and fluid taken from the sampling ports. The column had been in operation 21 months, so it was fully conditioned. There was no indication of crusting or ponding. The last virus had been applied one month earlier, and samples were taken to determine residual virus levels before the experiment commenced. Fill samples were taken just before dosing, and fluid samples shortly after dosing. Virus distribution in the fill is illustrated in Figure C-11, and fluid sample titers are shown in Figure C-12. Numerical data from the experiment are recorded in Table C-16.

Residual virus infectivity from previous applications was measurable in fluid samples down to 28 cm depth, but in fill samples only as deep as 21 cm. Titers decreased exponentially with depth in the fill samples. The fluid samples on day 1 (after dosing) displayed a different pattern. The virus was uniformly distributed from 6 to 28 cm depth, dropping off sharply below that point. On day 3, the fill titer at 6 cm depth had decreased by 50 percent but had increased at all depths below that down to 37 cm, suggesting that virus had relocated downward. Fluid sample titers had decreased at all depths to 28 cm, but at 37 cm were higher on day 3 than on day 1.

Samples taken on or after day 7 all showed lower titers than samples from day 3. This pattern continued through day 24, when the experiment was terminated. As shown by the percentages quoted in Figures C-11 and C-12, most of the virus inoculated did not penetrate the column as far as the sampling ports, at least in an infectious form.

Inactivation of virus retained in conditioned sand--The above experiment showed that migration of infectious virus through conditioned sand columns receiving moderate (5 cm/day) doses of STE was extremely limited. Furthermore, as time progressed, less and less of the infectivity could be recovered from within the column. The virus was clearly being inactivated in the column.

This inactivation was explored further using eight 3.5 cm minicolumns. They were packed with 20 g of conditioned sand and dosed with 0.7 cm of filtered STE/day for 10 days to restore the conditioning after packing. Each minicolumn was then inoculated with 1.0 x 10^7 PFU of Po-1 suspended in 0.7 cm of filtered STE. The following day, the fill from two of the columns was removed and

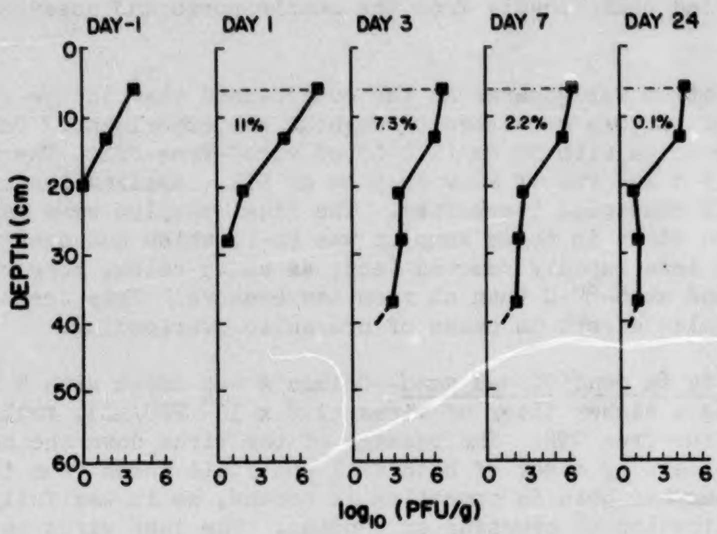

Figure C-11. Po-1 distribution in a conditioned sand column (fill samples). The percentage shown compare the amount of virus represented by the area within the curve to the total amount of virus that was applied in the dose on day 0.

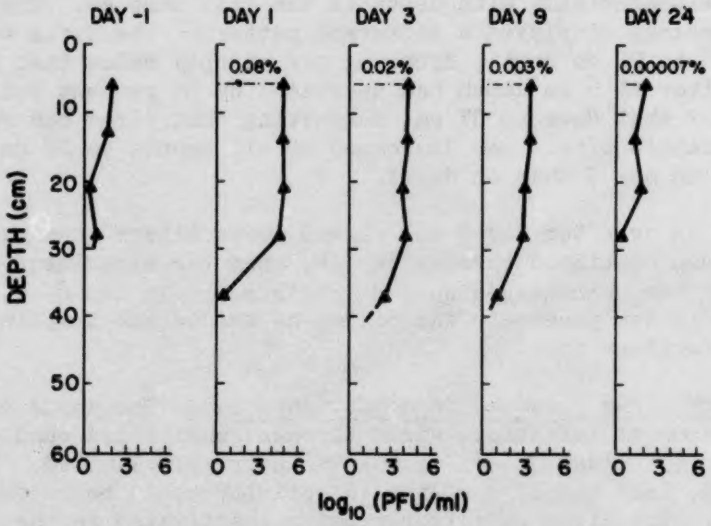

Figure C-12. Distribution of virus in a conditioned sand column (fluid samples). The percentages shown compare the amount of virus represented by the area within the curve to the total amount of virus that was applied in the dose on day 0.

TABLE C-16. TITERS OF SAMPLES FROM COLUMN A

Sample Type	Depth (cm)	Days after virus dose[1]					
		-1	1	3	7	9	24
Fluid	6	2.11[2]	4.95	4.36	--	3.59	2.08
	13	1.77	4.95	4.34	--	3.48	1.32
	21	0.3	5.00	4.00	--	3.23	1.87
	28	0.8	4.68	4.32	--	3.08	0.3
	37	0[3]	0.5	2.58	--	1.2	0
	44	--	0	0	--	0	--
	52	0	0	--	--	--	--
	60	--	0	--	--	--	--
Fill	6	3.67[4]	7.52	6.81	6.34	--	4.8
	13	1.89	4.90	6.38	5.76	--	4.45
	21	0.0	1.8	3.65	2.94	--	1.08
	28	0	0.8	3.59	2.52	--	1.51
	37	0	2-3?	3.04	2.36	--	1.45
	44	--	0	0	0	--	0
	52	--	--	--	--	--	--
	60	--	--	--	--	--	--

[1] Po-1 dose: 11.17 $\log_{10}$ PFU in 5 cm of STE; 8.24 $\log_{10}$ (PFU/mL).

[2] $\log_{10}$ (PFU/mL)

[3] No virus detected = 0

[4] $\log_{10}$ (PFU/gram)

stirred with 10 mL of fetal calf serum to elute the virus. The serum suspension was GA-8 filtered and assayed as were the effluents from the two columns. The other six columns each received 0.7 cm/day of virus free filtered STE. At 7, 14 and 28 days two more columns were sacrificed and assayed as above.

After 7 days the infectivity recoverable from the fill had dropped to 55 percent of the day 1 recovery; by day 14 it was 10 percent and by day 28, 2.5 percent. The effluent which accumulated in 7 days contained only 0.06 percent of the virus input; later accumulations were no higher. One column was dosed with ^{32}P-labeled virus; after 7 days only 0.2 percent of the label was recovered in the effluent. This indicates that the inactivated virus particles (or viral RNA, at least) were remaining in the columns.

To determine how disposal systems in sandy soils or mounds might be expected to perform during the winter, a set of eight minicolumns (similar to those just described) was run at 6 to 8° C. The duration of the experiment was extended from 4 to 8 weeks. The results presented in Figure C-13 include those of the room temperature experiment for comparison. The sorbed virus was not inactivated to any great extent in the cold, even after 8 weeks. After 4 weeks at room temperature, only 2.5 percent of the infectivity was recovered, whereas 57 percent of the virus was still infective after 4 weeks at 6 to 8° C. The cold columns also allowed approximately 10 percent of the virus input to pass into the effluent.

Virus inactivation rates in sand columns and fluid suspensions--The Po-1 retained in column A became inactivated at a daily rate of 0.088 $\log_{10}$ units, or 18 percent per day (from Figure C-11). The daily inactivation rate in 3.5 cm conditioned sand minicolumns was 0.062 $\log_{10}$, or 13 percent per day (Figure C-13). Virus in similar minicolumns operated at 6 to 8° C was inactivated at a rate of 0.0005 $\log_{10}$ (1.1%) per day.

Lefler and Kott (1974) studied the survival of Po-1 in fresh sand columns saturated with distilled water, tap water or oxidation pond effluent. The rate of Po-1 inactivation was 0.038 $\log_{10}$ (8.3%) per day with distilled water, 0.040 $\log_{10}$ (8.8%) per day with tap water, and 0.031 $\log_{10}$ (6.9%) per day with oxidation pond effluents. The experiments were run at 20° C, and samples were taken over

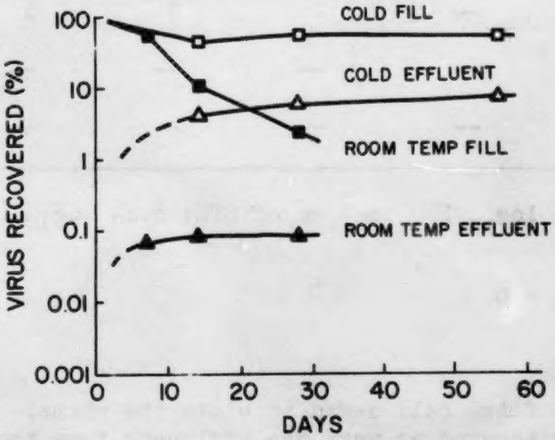

Figure C-13. Effect of temperature on inactivation of Po-1 in conditioned sand columns 3.5 cm deep.

a 175 day period. Columns at 4 to 8° C (fluid unspecified) had a Po-1 inactivation rate of 0.004 $\log_{10}$ (0.92%) per day. The authors pointed out that even drying did not inactivate all the virus. Sand columns to which Po-1 was sorbed were tested after 77 days of dryness, and 0.02 percent of the infectivity was recovered.

Rates of Po-1 inactivation in fluids at room temperature have been compiled from several of the experiments in the present study. Median daily rates were 0.146 $\log_{10}$ (29%) in STE, 0.237 $\log_{10}$ (42%) in column effluent or port samples, and 0.037 $\log_{10}$ (8.1%) in PBS or PBS - FC_2. At 6 to 8° C, the median daily virus inactivation rates were 0.047 $\log_{10}$ (10%) in filtered STE and 0.020 $\log_{10}$ (4.5%) in PBS. Inactivation rates were generally greater in STE and its fluid derivatives than in the columns.

Microbial role in the inactivation of sorbed virus--Cliver and Herrmann (1972) observed the inactivation of coxsackievirus A-9 when mixed with a pure culture of Pseudomonas aeruginosa. They did not observe inactivation with other viruses (including poliovirus) or other bacteria, but their study does suggest that microbial activity in the columns may have a role in virus inactivation.

To test this, six different types of bacteria were isolated (on the basis of colony morphology) from fill samples. Pure cultures were grown overnight in 0.4 percent nutrient broth powder (Difco) dissolved in filtered STE. The cultures were diluted 100-fold in the same medium, Po-1 was added, and the six flasks were agitated on a shaker at 30° C for 7 days. One additional flask was a bacteria-free control. The virus titer was reduced by approximately 2 $\log_{10}$ in all the flasks: The concentration in the control was 3.8×10^3 PFU/mL, and the cultures contained from 1.0×10^3 to 3.4×10^3 PFU/mL. These data indicate that the bacteria played a minimal role in the inactivation of the virus.

The question was explored further by comparing Po-1 inactivation in sterile and non-sterile minicolumns. Six Millipore cylinders were filled to 3 cm with conditioned sand. All openings were covered with foil and the columns autoclaved at 121° C for 30 min. After cooling, 1.0 cm of sterile phosphate buffer containing 2.2×10^7 PFU of Po-1 was added to each column; the columns were resealed and the virus allowed 2 days to adsorb to the fill. Three of the columns were then seeded with STE bacteria by dosing them with 2.0 cm of Whatman #1 filtered STE (F-STE). The three sterile control columns were dosed with 2.0 cm of sterile Whatman #1 filtered, autoclaved STE (the filtration was necessary to remove the precipitate induced by autoclaving). On subsequent days all six columns received 2.0 cm of sterile, filtered STE. To assure that sterile conditions were maintained in the control columns, the STE also contained antibiotics (0.1 mg gentamicin, 200 units penicillin, 0.2 mg streptomoycin, and 0.01 mg fungizone per milliliter).

After 33 days the effluents from all the columns were collected and tested for sterility in trypticase-soy broth. The samples from the seeded columns were not sterile, whereas all samples from the control columns were. The fill from each of the columns was removed and stirred with 10 mL of fetal calf serum to elute the virus. The effluents and the serum eluates of the fill were assayed for infectivity. There was no statistical difference (t-test at 95% confidence level) between the virus recoveries from the two sets of columns (Table C-17).

TABLE C-17. INACTIVATION OF SORBED Po-1 IN STERILE AND NON-STERILE 3 cm CONDITIONED SAND COLUMNS

Column	Recovery (%)	
	Fill	Effluent
Sterile	12	19
	22	19
	9	8
Non-sterile	13	6
	17	2
	12	3

There was no indication that the bacteria in the non-sterile columns caused a reduction in virus infectivity.

Adsorption of virus to floc formed in conditioned sand--When minicolumns were being filled with conditioned sand from larger columns, a light, flocculent material settled out of the slurry last and formed a layer on top of the sand. A series of experiments was undertaken to determine the role played by this floc in the retention of viruses by the columns.

Ten grams of conditioned sand were placed in a beaker, and 1.0 mL of ^{32}P-labeled Po-1 in phosphate buffer was added. After a 1.5 h adsorption period, the material was shaken with 10 mL or filtered column effluent to separate the floc from the sand grains. The floc was dried on a planchet and counted. It contained 93 percent of the labeled Po-1.

Next, a Gelman minicolumn was filled to 5 cm with fresh sand and dosed with filtered STE daily for 24 days for conditioning. The column was then dosed with 1.8 cm (1.2 void volumes) of ^{32}P-labeled Po-1 in phosphate buffer. This was followed by 31 daily 0.7 cm (0.5 void volumes) doses of virus-free filtered STE. The accumulated effluent was collected, and the fill was removed and shaken with two 10 mL portions of divalent cation-free, buffered physiological saline to separate the floc from the sand. Ninety-one percent of the labeled Po-1 was recovered in the floc and 4.0 percent in the accumulated effluent. Homogenization of the floc suspension in a Sorvall Omni-mixer released only 4.5 percent of. the bound virus.

To determine if virus adsorbs to floc in the absence of sand, a sample of floc was collected from conditioned sand removed from column B. The floc was packed into a small column (9 mm inside diameter, Pharmacia K9/15), and 4.7 cm (3.0 mL) or ^{32}P-labeled Po-1 was added. Less than 3 percent of the label passed through the 4 cm of floc. The column was frozen; and the cylinder of floc was removed and cut into slices which were dried, weighed and counted. Sixty-one percent of the virus was located in the top 16 percent of the column. The floc was strongly retentive of the virus and appears to be the primary site of virus adsorption in conditioned columns.

Summary

This study has examined the virology of septic tank waste treatment systems, with emphasis on the fate of virus during the soil phase of the treatment. Further study of the septic tank itself is merited: it seems to do a rather good job of removing fecal virus from wastewater.

Poliovirus type 1 in septic tank effluent (STE) was percolated through columns containing silt loam or sand. The silt loam removed the virus very strongly (not less than a 10-thousand fold reduction in 10 cm depth) and were unimpaired in efficiency after priods of use. Fresh sand also retained virus very efficiently; but after being "conditioned" by a few weeks' applications of STE, the sand was less retentive of virus, especially at temperatures as low as 6 to 8° C.

The assumed standard loading conditions in this study were 5 cm/day of STE applied to a 60 cm depth of sand. If the sand had been conditioned and was subjected to a ten-fold hydraulic surge overload, some virus was carried through the 60 cm of sand, especially at 6 to 8° C. Well over 99 percent of the virus was removed from the STE by the sand under the worst conditions that were devised and tested. Surge loading was not a factor in virus removal by silt loam because their low hydraulic conductivity precludes too-rapid infiltration of the STE.

Further applications of STE cause very little displacement of virus which has been retained in sand. The virus is gradually inactivated in the sand; the rate is lower at 6 to 8° C than at room temperature. The many bacteria present seem to have very little effect upon virus inactivation.

A floc which accumulates in conditioned sand appears to be the principal substance to which the virus adsorbs. Under standard loading conditions, virtually all the virus the STE might contain was removed by the conditioned sand during at least a period approaching 2 years.

In the design and operation of a slow sand treatment system, it is important to avoid large hydraulic surges or very uneven distribution of the waste, especially in cold weather. The results of this study indicate that virus reductions of 6 to 10 $\log_{10}$ are reasonable to expect of a properly designed and operated system.

THE FATE OF NUTRIENTS IN SOIL

The wastes from a household contain all the nutrients required for sustaining life processes. Disposal problems arise when high concentrations of these nutrients are discharged to relatively small areas. The nutrients which have the greatest potential of creating environmental or health problems are nitrogen (N) and phosphorus (P). This discussion will be concerned with reactions of these nutrients in the soil below septic tank-soil absorption fields. Water (the septic tank effluent) is the primary transport mechanism. Since vertical downward movement dominates in the unsaturated zone, the receiving body is the saturated zone or aquifer. The effect of this discharge on the aquifer is of importance, since the aquifer may be used as a drinking water supply source or may be eventually discharged as a surface flow.

There is no evidence that P in drinking water constitutes a threat to human health. Rather, P is of chief concern in relation to surface water quality, as it is one of the two major nutrients limiting algal productivity of lakes and impoundments. Because of the wide distribution and transformations of N in the natural environment, many fresh water lake eutrophication control strategies are based on P control, particularly where numerous point-sources of P represent a significant portion of the P budget in the lake.

Nitrogen, on the other hand, is a potential threat to public health. Nitrate (NO_3), the end product of nitrification which readily occurs under aerobic conditions in soils, and nitrite (NO_2), the intermediate form, can be toxic to humans particularly in infants, if ingested in excessive amounts. This toxicity arises from the hypoxia associated with the reaction of NO_2 with hemoglobin to reduce the oxygen carrying capacity of the blood. The NO_2 arises from microbial reduction of NO_3 in the human gut. With adults, lethal toxicity is virtually nonexistent, while clinical recognition of NO_3 toxicity in infants has resulted in essentially complete elimination of this problem. However, much less is known of the chronic toxicity (i.e., subclinical effects) of NO_3. Therefore, the U.S. Public Health Service recommendation of an upper limit of 10 mg/L of NO_3-N in potable water is almost certain to be retained for the foreseeable future (National Acad. Sci., 1972). Nitrate may also be toxic to livestock, but this appears to be of limited occurrence and is primarily associated with animal feed rather than water (Haz. Mater. Adv. Comm., 1973).

Recently a number of N-nitroso compounds have been found to be carcinogenic to laboratory animals, and man is probably also susceptible (Tannenbaum, 1976). This class of compounds, commonly referred to as nitrosamines are formed from the reaction of NO_2 with secondary amines. Formation of significant amounts of these compounds in the natural environment has not yet been demonstrated, but they are resistant to microbial attack (Tate and Alexander, 1976). A more likely mode of human exposure is formation in the body by reaction of ingested NO_2 with amines in the stomach (Tannenbaum, 1976). Saliva contains significant NO_2, which increases markedly on ingestion of NO_3. Tannenbaum (1976) indicated that NO_3 in food and water represents a high carcinogenic potential and noted that there is strong epidemiological evidence in several countries that high NO_3 intake is casually related to gastric cancer.

Background

Phosphorus Transformations--

Phosphorus in household wastes--Phosphorus in septic tank effluent originates mainly from detergents with phosphate builders and from human excreta. The relative contribution of detergent P will vary with the amount of detergent used and its P content, but Sawyer (1965) has estimated that detergent-based P accounts for about 50 to 75 percent of the total P in domestic wastewater. The contribution from human excreta has been estimated by Sherman (1952) to range from 0.23 to 1.05 kg per person per year with a mean of about 0.6 kg. These estimates were based on dietary data. The results of the raw wastewater characterization phase of this study (Appendix A) indicated the collective contribution of P in bathing, clotheswashing and dishwashing wastewaters to be approximately 3.4 g/cap/day (86 percent) with toilet usage contributing 0.6 g/cap/day (14 percent).

The anaerobic digestion which occurs in the septic tank, converts most of the organic and condensed phosphate forms, to soluble orthophosphate. Magdoff, et al., (1974b) and Otis, et al., (1975) found more than 85 percent of the total P in most septic tank effluents to be in this form. The relatively small amounts of organic P and also the condensed phosphates present in many septic tank effluents will eventually be converted to orthophosphate. The condensed phosphates such as meta-, pyro-, and tripolyphosphate will react with soils in a manner similar to orthosphosphate (Black, 1970). Magdoff, et al., (1974b) found that the total P concentration in the septic tank effluent used in their column studies ranged from 15.6 to 24.5 mg/L with a mean value of 20.6 mg/L. Otis, et al., (1975) in detailed studies of 6 septic tank systems, found average effluent concentrations of total P to range from 11.0 to 31.4 mg/L with a median value of about 12 mg/L. This concentration, coupled with the measured average flow rate of 182 liters per person per day gives the total of 0.8 kg of P per person per year.

Chemisorption reactions--At low P concentrations (< 5 mg/mL-P) in the soil solution at equilibrium, the phosphate ion becomes chemisorbed on the surfaces of Fe and Al minerals in strongly acid to neutral systems, and on Ca minerals in neutral to alkaline systems. The pH of septic tank effluent is nearly neutral. Viraraghavan and Warnock (1974), and Walker (personal communication) found pH values of non-calcareous sandy soils under seepage fields to range from 6.2 to 7.0. In this pH range, all of these metal cations (Fe^{+++}, Al^{+++}, Ca^{++}) could probably participate in P immobilization reactions, with Fe and Al-bound P dominating in non-calcareous soils and Ca-bound P in calcareous soils.

Precipitation reactions--As the P concentration in the soil solution increases, there comes a point where one or more phosphates precipitates may form. This point can be predicted from the ion activity products (solubility products) if all of the relevant ion activities are known. Ion activity products of some of the more important compounds are given by Lindsay and Moreno (1960). These compounds include strengite ($FePO_4 \cdot 2H_2O$), variscite ($AlPO_4 \cdot 2H_2O$), dicalcium phosphate ($CaHPO_4 \cdot 2H_2O$), octacalcium phosphate [$Ca_4H(PO_4)_3 \cdot 3H_2O$], and hydroxyapatite [$Ca_{10}(PO_4)_6(OH_2)$]. In acid soils, most of P sorption involves the Al and Fe compounds while in calcareous or alkaline soils, Ca compounds predominate.

In the pH range encountered in septic tank seepage fields, hydroxyapatite is the stable calcium phosphate precipitate. However, at relatively high P concentrations similar to those found in septic tank effluents, metastable compounds such as octacalcium phosphate are formed initially, followed by slow conversion to hydroxyapatite (Lindsay and Moreno, 1960).

Phosphorus reactions under anaerobic conditions--Although the phosphate ion itself is not chemically reduced under redox conditions normally occurring in nature, subjecting a previously well-aerated, non-calcareous soil to reducing conditions will almost invariably result in an increase in dissolved inorganic P. This is to be expected since much of the P in soils is bound to ferric iron, which is converted to the soluble ferrous form under reducing conditions. The P is then released to the soil solution where a new equilibrium is established with the Al- and/or Ca-bound phosphates. Patrick (1964) found that extractable P increased rapidly as the redox potential decreased from +200 to -200 mv. This is the same redox potential range over which ferric compounds are reduced.

Magdoff, et al., (1974b) found different results in soil columns dosed with septic tank effluent. The soil used in this case contained free $CaCO_3$, and they postulated that the lower P values in the leachates after the columns crusted and became anaerobic were due to increased Ca ion concentrations resulting from dissolution of $CaCO_3$ by organic acids formed under the anaerobic conditions. The P concentrations in the leachates were very close to those predicted from the solubility product of octacalcium phosphate.

Time dependence of P immobilization--The rate at which P is sorbed from solution onto the surfaces of soils and soil constituents has been shown to consist of a rapid initial reaction followed by a much slower reaction which appears to follow first order kinetics. Kuo and Lotse (1973) reported that 80 percent of the P sorption by $CaCO_3$ was completed within 10 seconds, while Chen, et al. (1973), found that the initial reaction on kaolinite required about 24 hours. Chen, et al. (1973), found that the slow sorption reaction continued for several weeks. This slow reaction has been attributed to diffusion of P into the sorbing material (Scholten, 1965), or a slow decomposition-precipitation reaction (Hsu and Rennie, 1962; Kuo and Lotse, 1973; Griffin and Jurinak, 1974). Soil material directly under the absorption bed will be exposed to relatively high concentrations of P throughout the life of the system, so that this slow reaction is important in determining the amount of P immobilized.

Capacity of soil to immobilize P--Investigators attempting to describe P movement in soils have utilized models involving: 1) a first order kinetic equation, 2) a series of exponential terms, 3) a second order kinetic equation based on the Langmuir adsorption isotherm, 4) the Elovich equation, 5) a kinetic equation based on the Freundlich adsorption isotherm, and 6) mass transfer (Kuo and Lotse, 1973; Novak and Adriano, 1975). All of these equations predict a relatively sharp boundary between a zone of near-maximum P adsorption, extending down from the surface or layer of introduction, and the underlying soil. The coefficients for the equations have usually been derived from laboratory experiments involving relatively short equilibration times. As a result, the slow decomposition-precipitation reactions which should occur with the high P concentrations in septic tank effluent are not taken into account, and maximum immobilization values are underestimated. Adriano, et al. (1975), studied the P retained by soils subjected to wastes from a food processing plant and a dried milk and cheese plant. They concluded that the amount of P retained by the soils greatly exceeded the Langmuir adsorption maximum, and attributed the excess to precipitate formation, particularly of Ca phosphates. Sawhney and Hill (1975) also found that soils retained P in excess of the Langmuir adsorption maxima, and that the P sorption capacity of a soil could be rejuvenated by drying and wetting the soil following saturation with P. This procedure apparently exposes more sorption sites probably as a result of precipitate nucleation and migration of sorbed P to the precipitate nucleation sites. Similar results were obtained in the Netherlands by Beek and deHaan (1974).

Nitrogen Transformations--

Nitrogen in household wastes--Ammonium N and organic N constitute the most prevalent forms of N in household wastes (dissolved N_2, though always present, is not considered here). Urine is the major source of N, and this N is largely in the forms of urea, uric acid and creatimine. The remainder of the N is largely in undigested foodstuff and bacterial cells (HMAC, 1973). The results of the raw wastewater study (Appendix A) indicated an average daily nitrogen contribution ranging from 6.1 to 16.8 g/cap/day. In a daily flow of 170 L/cap/day this yields a concentration of approximately 36 to 94 mg/L-N. Effluents from 6 septic tanks showed a range of 26 to 76 mg/L-N (Appendix A).

Septic tank and seepage bed--Anaerobic conditions prevail in the septic tank, and the soluble N compounds of low molecular weight are mineralized rapidly to NH_4 by enzymatic and microbial degradation. The N in the effluent as it leaves the tank is about 75% NH_4-N and 25% organic N (Otis, et al., 1975) with a C:N ratio of about 10. Some of the N is associated with the sludge solids and thus remain in the tank, although mass balance studies indicate this is a small fraction of the total. Particulate N leaving the tank is retained at the seepage bed-soil interface to become part of the crust layer. This N will be slowly mineralized to NH_4-N.

Below the seepage bed--Since crust formation approaches an apparent steady state condition, additions and removals of N are essentially equal. Thus, in a stable system, N input to the soil below the seepage bed is equivalent to N output from the tank, with little or no net N removal. When the bed is crusted or if effluent is added by dosing, the normally underlying soil will be aerobic and moist. Temperature will vary depending on the season, but even in mound systems, is always above freezing in the winter. Thus, under these conditions, nitrification will occur within the underlying soil.

Optimal nitrification in soil occurs when the soil moisture tension is in the range of 300 cm to 100 cm (Sabey, 1969; Stanford and Epstein, 1974). Bouma (1975) found tensions of 60 to 100 cm in sandy loams, loamy sands and loams and 30 to 35 cm in finer-textured soils under properly functioning seepage beds. Thus, finer-textured soils will have less air-filled pore space and the possibility of zones with anaerobic conditions exists. These anaerobic areas may be the site of some denitrification. Since soil textures are rarely homogeneous vertically, moisture tension discontinuities at textural boundaries also could result in zones of high water content and subsequent anaerobic conditions.

Because the effluent is under anaerobic conditions in the tank and seepage bed, there is no opportunity for nitrification until the effluent reaches the soil. Thus, the inorganic N is present only as NH_4-N on the soil cation exchange sites and in solution. The continuing input of cations (Ca^{++}, Mg^{++}, Na^+, K^+ and NH_4^+) from the effluent will maintain a constant ratio of NH_4-N in solution to NH_4-N on the exchange sites. Therefore, NH_4-N will move downward with the percolate once equilibrium of the effluent with the cation exchange sites is achieved. Unless the soil is submerged or oxygen diffusion into the soil is limited by some other means, aerobic conditions will prevail in the unsaturated zone within a very few cm below the bed. At this point nitrification will commence. The point at which complete nitrification of the NH_4-N occurs will be a function of the contact time of the effluent and the rate of nitrification. Factors which affect the rate of nitrification include pH, aeration and moisture conditions, number of nitrifiers, and temperature (Alexander, 1961). The pH of the soil beneath the seepage bed is near neutrality (Walker, personal communication) which is optimal for nitrification.

Denitrification would be desirable, since it is the only feasible mechanism of N removal. However, energy (available organic carbon) is required. The only sources of organic matter under the bed are organic matter native to the soil and that from the effluent. Since subsoils are typically low in organic matter and organic matter from the effluent is rapidly oxidized under the seepage bed (Magdoff, et al., 1974a), denitrification would not be expected to be a significant N removal mechanism.

Thus, as a first approximation, all of the N leaving a household can be expected to eventually exist as NO_3-N under the seepage bed. Nitrate is not retained by soils, and it will move with the percolate to the groundwater. The resulting groundwater NO_3-N concentration and ultimate fate of this N is dependent on the hydrogeology of the region and other sources and sinks of N.

Previous Investigations of Groundwater Pollution by N and P from Subsurface Seepage Beds

Woodward et al., (1961) reported on an extensive survey of over 63,000 private water supply wells in 39 communities in Minnesota. The majority of these wells were shallow, and most of the communities were served by individual septic tank systems. Forty-eight percent of the wells sampled had NO_3-N concentrations above background levels and 11 percent had concentrations above 10 mg/L-N. The authors explained the variable results on the basis of soil profile properties, well depth, population density and hydrogeology. One community, which was located in an area where there was a continuous clay layer in the soil profile, had no ground water contamination. A general relationship of increasing water quality with well depth also existed. More contamination was found in older communities (pre-1940) than in post World War II communities. The reasons for this were not clarified in their study. Crabtree (1972) found that 15 percent of the wells in an area in central Wisconsin where the population was served exclusively by septic tank systems had greater than 10 mg/L of NO_3-N, and Miller (1972) found that groundwater in an urbanizing sandy soil area of Delaware contained 10 to 30 mg/L of NO_3-N.

Several investigations have been reported which verify the rapid nitrification of septic tank effluent in sandy soils. Robeck, et al., (1964) obtained 80 percent nitrification of added NH_4-N in 150-cm soil lysimeters, while de Vries (1972) and Pruel and Schroepfer (1968) found essentially complete nitrification within 30 to 60 cm.

Dilution by uncontaminated groundwater is the only significant mechanism of lowering NO_3-N concentration in the groundwater below seepage beds overlying aerobic soils. Polta (1969) found a rapid decrease in NO_3-N concentration in groundwater with increasing distance from a septic tank-dry well system, with NO_3-N concentrations decreasing from 30 mg/L directly under the system to 5 mg/L at 43 m down gradient. Pruel (1966) obtained data from septic systems on sandy soils, which showed that about a 30 m distance down-gradient from the seepage bed was required to lower NO_3-N below 10 mg/L. Similar findings were reported by Childs (1973), although Polkowski and Boyle (1970) found that 20 m was required for the system they evaluated.

While NO_3-N usually is the major N form found in groundwater under septic tank absorption fields, high concentrations of NH_4-N have been reported occasionally. This has usually occurred where the depth of the aerobic zone between the bed and the groundwater was shallow (< 0.6 m). Ammonium-N concentrations should also increase in winter when low temperatures slow the rate of nitrification and thus effectively increase the distance required for complete nitrification. Also, conditions under which saturated flow might occur (such as in a new system or one which did not have a clogging mat) could permit movement of NH_4-N.

Chloride (Cl^-) moves through soils much like NO_3-N, and is present in household wastes at concentrations much higher than background levels. Thus, its presence can also indicate the rate and direction of effluent movement. Dudley and Stephenson (1973) noted consistently high correlations between NO_3-N and Cl^- concentrations in groundwater below a number of seepage beds. They investigated 11 sites in Wisconsin and found significant Cl^- contamination at all sites. The average NO_3-N concentration in the groundwater below systems installed in sands was 15 mg/L, and a distance of about 15 m down-gradient from the system was required before the concentration decreased to less than 10 mg/L. Significant NH_4-N concentrations occurred at two sites, one located on impermeable glacial till soil, and the other on a site with a high water table.

Ellis and Childs (1973) investigated 19 septic tank sites on sandy soils around Houghton Lake, Michigan, and found significant NO_3-N input to the groundwater from 6 of these sites. Significant concentrations existed from 30 to 100 m from the sources and generally affected the upper 2.4 m of the groundwater aquifer.

Several reports have noted a high degree of variability in results of groundwater sampling, both in terms of time and space. This can be explained by a number of factors. A groundwater mound usually develops under the system (Bouma, et al., 1972; Dudley and Stephenson, 1973) which can markedly affect the flow pattern. Also, different usage patterns and temperature effects on the rate of nitrification result in differing concentration patterns of NO_3-N in the groundwater.

Several cases of significant P contamination in groundwater below seepage beds have been reported. Ellis and Childs (1973) found significant P movement (up to 30 m) from a number of systems, with PO_4-P concentrations ranging from 2 to greater than 20 mg/L for up to 1.2 m into the groundwater aquifer directly under the systems. Phosphorus was moving into a nearby lake from a number of these systems. Dudley and Stephenson (1973) found significant concentrations (> 1 mg/L total P) above background levels (< 0.2 mg/L total P) in 4 of the 11 sites they examined. Three of the sites were in coarse-grained outwash sands and gravels, but one was in an impermeable glacial till. None of the newer systems had significantly increased concentrations of P in the groundwaters immediately below them.

Experimental Approach

Results of the project research activities designed to evaluate the fate of P and N from subsurface disposal of septic tank effluent are summarized below, as related to other research and to the potential for contamination of groundwater.

Evaluation of Existing Systems in Sands--

Five study sites, located in permeable sandy soils in central Wisconsin were investigated. Details of the methodology have been reported by Walker, et al., (1973a; 1973b). The forms and amounts of N in the soil under a seepage bed were determined from samples obtained by excavation adjacent to each system. Groundwater samples, direction of flow and gradients were obtained using observation wells.

Columns Representing Fill Type Disposal Systems--

Detailed description of this research is given by Magdoff, et al., (1974a; 1974b) and Magdoff and Keeney (1975). Large polyvinyl chloride (PVC) columns (14.7 cm inside diameter) were filled with soil materials to represent those used in a mound (See Figure C-14). The fill material consisted of either a sand or a sandy loam subsoil. The columns were equipped with tensiometers, sampling ports, air entry ports and redox electrodes. They were loaded with 8 cm/day of effluent from a household septic tank. Room temperature was about 15° C.

Monitoring of Existing Fill Systems--

Concentrations of N and P in experimental fill systems were determined by *in situ* sampling of the liquid. One mound was designed for use in areas underlain by creviced bedrock (Bouma, et al., 1974c). The others were designed for use on slowly permeable soils (Bouma, et al., 1975b).

Analytical Procedures--

Ammonium-N and NO_3-N concentrations in water samples or 2N KCl extracts of soil samples were analyzed by the steam distillation procedure of Keeney and Bremner (1965b). Organic C was measured as described in Standard Methods (1965) and total N by Kjeldahl (Bremner, 1965a). Chloride was determined by the procedure of Cotlove, et al., (1958).

Oxygen, N_2 and CO_2 were determined on a Barber-Coleman Model 23-P gas chromatograph (Chen, et al., 1972) and methane was determined on a 407 Series Packard gas chromatograph with a flame ionizer detector (Macgregor, et al., 1973). Platinum electrodes for Eh measurement were prepared by fusing 0.6 cm segments of a 20-gauge Pt wire to a 12-gauge solid Cu wire. After replacing the original rubberized outer sheath of the Cu wire, the Cu-Pt junction was coated with epoxy cement to give a water tight seal. The electrodes were replatinized with black platinum and checked according to the procedures of Quispel (1946).

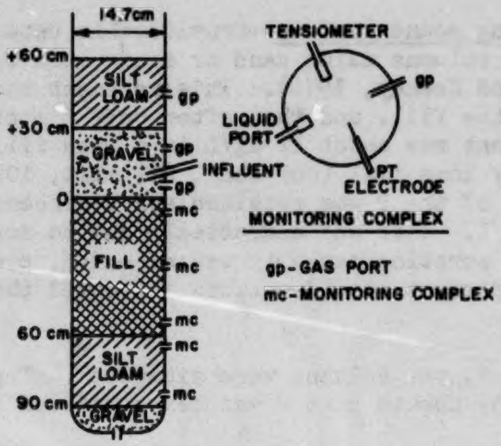

Figure C-14. Column design for mound simulation

Results

Phosphorus--

Existing systems in sands--The results of monitoring studies of P concentrations in groundwater under and adjacent to existing systems in sands were reported by Bouma, et al., (1972). Two of the 5 systems studied had relatively high groundwater concentrations of P. One of the systems was 12 years old, and the other had been functioning for 8 years. Both had coarse sands and gravels between the seepage system and the water table. Background PO_4-P ranged from less than 0.02 to 0.03 mg/L, and PO_4-P in wells just adjacent to the systems ranged up to 9.5 mg/L. Phosphorus above background was not found in groundwater under a new system (< 1 year old) or under the systems which had an intervening layer of clay between the system and the aquifer. These findings largely confirm those reported in the literature.

Existing mound systems--Three experimental mound systems, based on preliminary designs, were monitored in 1970 and 1971 (Bouma, et al., 1972). Well water samples were obtained in the vicinity of one of the mounds which was located on a somewhat poorly drained sandy loam. One of the wells adjacent to the system had elevated PO_4-P concentrations (0.37 mg/L). The fill for this system was sand, and the groundwater was less than 1 m below the mound. No other groundwater P data under existing mounds are available. However, P concentrations of percolate within the fill of several existing mounds was determined. Phosphorus (PO_4-P) concentrations generally ranged from 0.2 to 0.6 mg/L at the fill-soil interface (Bouma, et al., 1972). Phosphorus was also measured in the soil solution in a sandy loam fill system installed over creviced bedrock (Bouma, et al., 1974c). The data showed that, after 18 months of operation, total P in solution was about 8 mg/L, indicating that the P removal capacity of the system was low.

Columns representing mound systems--Considerable data are available on P retention in laboratory columns using sand or sandy loam fill (Magdoff, et al., 1974a; 1974b; Magdoff and Keeney, 1976). This research showed that P was not completely retained in the fill, and that after only 3 months of loading, P concentration in the effluent was about 12 mg/L in a sand fill (100% sand) and 2 mg/L in calcareous sandy loam fill (68% sand, 22% silt, 10% clay) (See Table C-18). Initially, most of the P was retained and P concentrations in the effluent were less than 1 mg/L. This was undoubtedly due to sorption of P by soil constituents. Once the sorption capacity was exceeded, precipitation reactions dominated. Solubility product considerations indicated that octacalcium phosphate was being formed.

After the experiment, the columns were dissected. The total P distribution profile (See Figure C-15) showed that P was being removed in the sand fill and in the underlying soil.

Nitrogen--

Existing systems in sands--The results of a monitoring study on existing systems located in sands in central Wisconsin were reported by Walker, et al., (1973a; 1973b). The findings confirmed other reports that nitrification will readily occur if 1 to 2 m of unsaturated soil are present above groundwater. Table C-19 summarizes the findings and indicates that nitrification was complete at 5 to 15 cm below the clogging mat.

The effect of septic tank systems on NO_3-N in the groundwater was also evaluated (Walker, et al., 1973b). The results demonstrated the heterogeneity of the groundwater regime, even in an area with sandy soil profiles. Results for three of the systems are given in Figures C-16, C-17 and C-18. Below System 1 (Figure C-16), both NH_4-N and NO_3-N were found in the groundwater. This was a large system, but loading from the household decreased markedly during

TABLE C-18. PHOSPHORUS CONCENTRATIONS IN SOLUTION IN UNCLOGGED SAND AND CLOGGED SANDY LOAM COLUMNS

		Total P in solution (mg-P/L)	
	Depth (cm)	Unclogged[1] Sand	Clogged[2] Sandy loam
Influent		23.9	18.3
	5	22.1	14.5
Fill	30	21.6	13.7
	55	17.0	8.2
	65	9.8	1.0
	83	11.0	1.2
Effluent		11.8	1.6

[1] After 94 days of loading.
[2] After 73 days of loading.

TABLE C-19. NO_3-N CONCENTRATIONS IN UNSATURATED SOIL SOLUTIONS[1] AT SELECTED DEPTHS BELOW SEEPAGE BEDS (Walker, et al., 1973a)

Depth Below Crust		System 1	System 3	System 4	System 5
cm		mg/liter			
5		20	50	100	130
15		50	160	70	120
45		50	140	70	80
75		50	80	80	90
	Avg.	43	110	80	105

[1] The concentrations (µg/g dry soil) of NO_3-N in unsaturated soil solutions were obtained on field-moist samples.

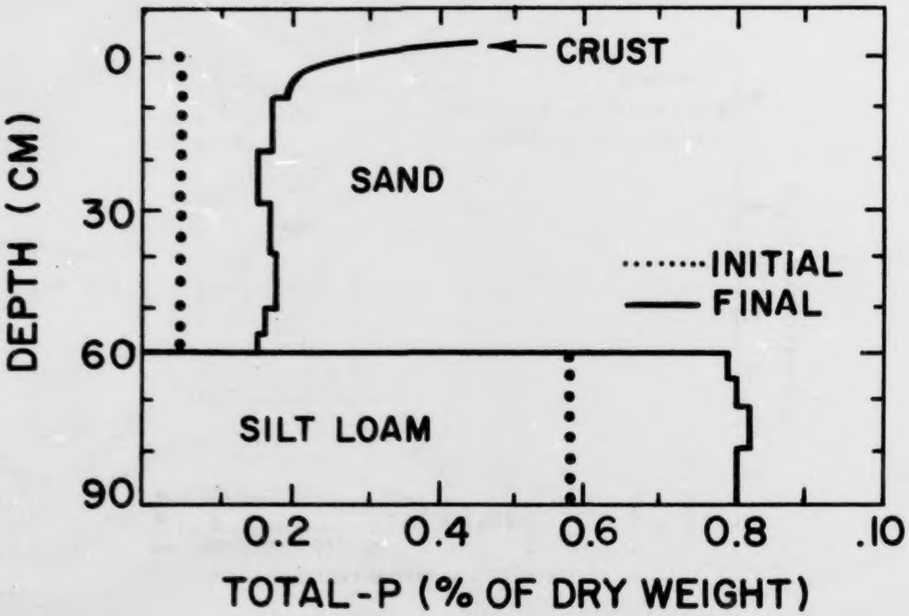

Figure C-15. Total phosphorus profiles in laboratory columns.

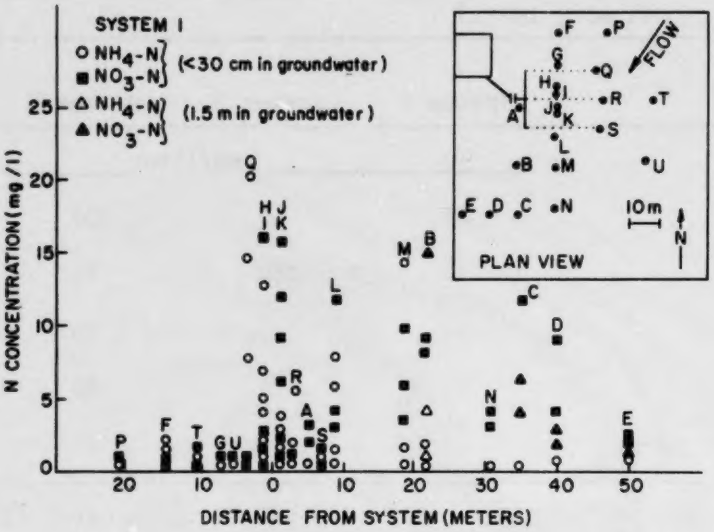

Figure C-16. Concentrations of NH_4-N and NO_3-N in ground water as a function of distance to the seepage bed (Walker, et al., 1973b).

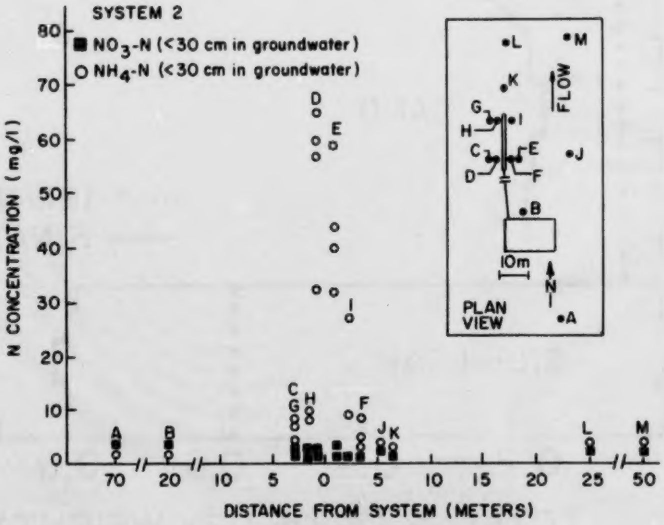

Figure C-17. Concentrations of NH_4-N and NO_3-N in ground water as a function of distance to the seepage bed (Walker, et al., 1973b).

the study. The bed was probably aerobic during periods of low loading, and, when used, the effluent percolated so rapidly through the soil that nitrification did not always occur. System 2 (Figure C-17) was constructed in the groundwater and, therefore, little nitrification could occur. Another system not shown was constructed over a clay layer 8 m below the bed. Groundwater below this layer had little NO_3-N indicating that the clay layer acted as a barrier to vertical water movement immediately below the system, causing the percolating waste to move along the layer and emerge or percolate downward at some other place. System 4 (Figure C-18) provides a classical picture with maximum NO_3-N (about 40 mg/L) directly adjacent to the bed. The NO_3-N concentration was still about 10 mg/L at 70 m from the bed.

Existing mound systems--From investigations at one mound system, Bouma, et al., (1974c) found that nitrification was complete within the fill (0.7 m of fine sand). Influent total N to this system averaged 62 mg/L. There was 66 mg/L of NO_3-N in the liquid reaching the interface between the fill and the silt loam topsoil and 54 mg/L of NO_3-N 20 cm below the interface, indicating some NO_3-N retention (possibly denitrification) by the topsoil. Ammonium-N decreased to below detectable levels. Interestingly, the NH_4-N concentration 20 cm below the interface in the winter was 8 mg/L. During this sampling period, NO_3-N was 30 mg/L, and organic N was 10 mg/L. By May, this had changed to 3 mg/L of NH_4-N, 57 mg/L of NO_3-N, and 0.3 mg/L of organic N. Thus, mineralization and nitrification were inhibited by the low temperatures in the middle of winter.

Columns representing mound systems--Results of this study (Magdoff, et al., 1974b) also showed that nitrification in the sand fill is rapid and complete, and that some denitrification may be occurring at the fill-topsoil interface where nearly saturated conditions exist (Magdoff, et al., 1974a). After the experiment, the columns were sampled for total N. The results show that, with the exception of the clogging mat, very little N was retained in the sand fill, and that N was mineralized and removed from the silt loam topsoil (See Figure C-19). Similar results were obtained for the distribution of carbon.

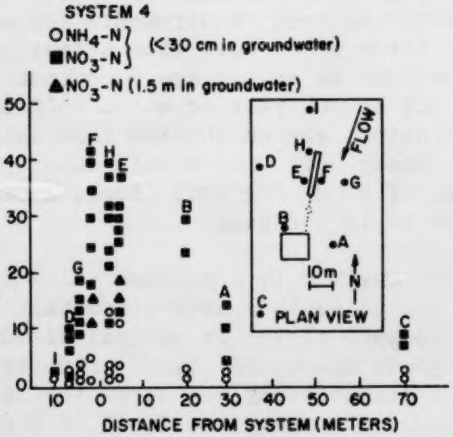

Figure C-18. Concentrations of NH_4-N and NO_3-N in ground water as a function of distance to the seepage bed (Walker, et al., 1973b).

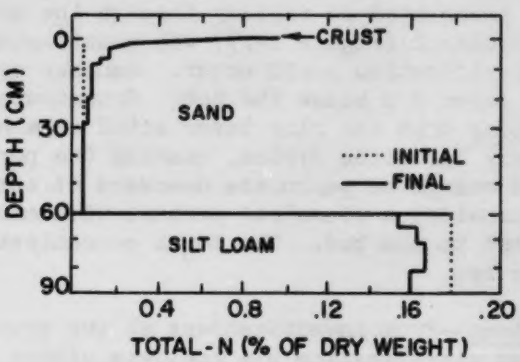

Figure C-19. Total nitrogen profiles in laboratory columns.

Discussion

Phosphorus--

Results of this project and of other investigations have shown that septic tank soil absorption systems, whether conventional or mounded, can represent a significant source of P to the local ground water system. However, P is tightly sorbed by most soils (Black, 1970), and significant inputs to surface water would be likely only if the soil had a very low sorptive capacity, e.g., sand, and the system were in close proximity to surface water.

The depth of penetration of the P "saturated" layer can be calculated if the P loading rate and the P immobilization capacity of the soil are known. The extreme case would be a seepage field in a sandy soil. The Wisconsin State Board of Health minimum soil absorption area requirements for a 3 bedroom house are about 21 m^2 of bottom area for a sandy soil with a fast percolation rate. If an effluent input of 727 liters/day at an average P content of 12 mg/L is assumed, the total P load would be 3.2 kg per year or about 1500 kg/ha per year (The input and median P concentrations are calculated from data of Otis, et al., 1975 for a family of 4). For a sandy soil with a bulk density of 1.6 g/cm^3, and a Langmuir adsorption maximum of 90 µg P/g soil (Peck, 1962) the soil would be P-saturated to a depth of 104 cm in one year.

The Langmuir adsorption maximum is a minimum value for potential P immobilization. Walker (personal communication) determined that P extracted from sandy soils beneath septic tank seepage fields in central Wisconsin ranged from about 100 to about 300 µg/g. Magdoff and Keeney (1976) reported immobilization of 121 µg P/g by a sandy soil in a column study which ran for less than a year. Based on a P immobilization value of 200 µg/g, the depth of P penetration would be about 52 cm per year in the hypothetical system. The depth of penetration would be less on finer textured soils because the maximum P immobilization values would be higher, and the loading per unit area would be less as larger absorption areas would be required. Peck (1962) found that the average Langmuir adsorption maximum for silt loam surface soils in Wisconsin was 186 µg/g. If such a soil has a relatively slow percolation rate, an absorption bed of about 70 m^2 would be required to handle the volume of effluent used in the hypothetical sand system. Thus, the areal P loading rate would be decreased to about 0.3 that of the sand, and the adsorption maximum would be increased by a factor of about 2.1. This would result in P penetration of about 15 cm per year if only the adsorption maximum values were considered, and less than 10 cm per year if additional immobilization from precipitation were assumed. The data of Magdoff and Keeney (1976) would suggest that P immobilization in excess of the adsorption maximum is significant on silt loam soils, as they found 307 mg/g-P immobilized compared to the mean adsorption maximum value of 186 µg/g obtained by Peck (1962).

This analysis supports the experimental findings that P movement is the most extensive on coarse sandy soils. If sufficient systems are located adjacent to a lake or impoundment and sandy soils predominate, P removal from the effluent would seem necessary to protect the water quality of the lake.

Nitrogen--

Nitrogen loading to ground waters from septic tank disposal fields is a certainty in nearly all cases. The seriousness of the situation is essentially a function of the hydrogeology of the area and the level of background contamination. The movement and mixing of nitrate in the ground water is complicated, and prediction of problem areas is difficult. Problems are most likely to exist in areas with higher housing densities (Walker, et al., 1973b). Even in these situations, proper well construction (deep casing) might be sufficient to negate or reduce NO_3-N contamination of drinking water. The uncertainties about the severity of NO_3-N toxicity, and the lack of documented evidence of the relative contribution of septic tank-soil absorption systems to the NO_3-N contamination of ground water would indicate that, at this time, there is not a widespread need for NO_3-N removal systems. However, continued monitoring of areas with relatively high development densities employing this form of wastewater disposal, especially in coarser soils, is recommended as a prudent course of action by health authorities.

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APPENDIX D

INSTITUTIONAL AND REGULATORY ASPECTS OF TREATMENT AND DISPOSAL OF SMALL WASTEWATER FLOWS

CURRENT REGULATIONS

The typical means of implementing control of the various on-site sewage treatment and disposal systems in use today is via a regulatory program which imposes requirements and/or restrictions upon the systems and upon those who design, install, own or operate them. While the regulatory programs used in different jurisdictions appear to vary greatly, there are many basic similarities (Patterson et al., 1971). This variety of programmatic approaches exists due to preferences for (or against) certain departments and agencies, as they exist in any municipal or state government. The existing local departments and/or agencies which typically have responsibility for supervision of on-site systems vary extensively including: departments of health, plumbing, building, development, planning, zoning, environmental protection, public works and drainage and conservation commissions. (For convenience, the terms municipal and municipality will be used here to mean all general purpose governmental units, i.e., city, town, village, borough, county, etc., and any special purpose districts which are empowered to control on-site sewerage.)

The institutional approach also varies with the degree that on-site sewerage is perceived as a problem and with the extent and nature of regulation to which the citizens are accustomed. The emphasis of this initial discussion of regulatory programs centers almost exclusively upon local municipal and state programs; because, aside from surface water discharge from on-site sewage treatment systems, there are no federal statutes or regulations which are directly applicable to these regulatory programs.

Each local or state program contains common elements such as: 1) the type of regulations, and 2) the regulatory or enforcement mechanisms used. These two elements will be discussed briefly below.

Type of Regulations

Regulations for the control of on-site sewerage may be effected in a number of ways. These include the use of specification standards; performance standards or regulations which indirectly control on-site systems, i.e., land use controls, subdivision regulations, etc.

Specification Standards--

Design standards establish detailed requirements and/or restrictions for specific components or location of on-site sewerage systems. This type of regulation is found in almost all regulatory programs, i.e., minimum septic tank volume shall be 750 gal, soil absorption field shall be set back 50 ft from any water course, etc. However, these standards have been found to vary widely from one regulatory program to another (Plews, 1977). This is especially so when comparing one state's program to another. One must logically assume that those individuals regulated by these programs and those involved in the regulatory programs could reasonably question this variance, since all programs are intended to have the identical purpose, the protection of public health. However, despite this singular purpose, the fact remains that difference in specification standards does exist.

Additionally, the use of specification standards has often been objected to by the designers of these on-site systems because the standards deny any opportunity to truly "design" the system for the existing situation. This objection is quite valid in those instances where a competent designer is, in fact, stymied by the codification of the design parameters of an on-site system. However, in fairness, the use of specification standards has been defensible because the previous designers of these systems did not understand or were not concerned with proper design of these systems (Winneberger and Klock, 1973).

Performance Standards--

These standards establish requirements for the performance of on-site sewerage systems without specifying how such standards are to be met. Performance standards generally are not a type of regulation used in on-site system regulatory programs. Examples of performance standards are: effluent standards, equipment standards and operation standards. Systems which discharge effluents to surface waters have effluent standards as established by P.L. 92-500 and the regulations adopted pursuant to the act. Since these standards simply impose effluent limitations without establishing specific requirements for design, they are clearly performance standards. The National Sanitation Foundation's (1970) NSF Standard No. 40 for individual aerobic wastewater treatment plants is an example of equipment performance standards. Implicit in all regulatory programs is the principal purpose of protection of public health. Often, this purpose is expressed or implied as a performance standard by prohibiting the creation or maintenance of a nuisance or unhealth condition. While nuisances are quite difficult to prove in legal actions, on-site systems typically are subject to this operational standard.

Indirect Controls--

Land use or zoning controls may result in indirect or de facto regulation of on-site systems. For example, in Wisconsin, state regulations for unsewered subdivisions impose minimum lot size and soil characteristic requirements upon nearly all land parcels created in the state. As a consequence of this direct regulation of the land subdivision process, on-site system controls are strengthened by assurance that building lots will be adequate for an on-site system. Similarly, a unit of government could effect indirect control over on-site systems by adoption of comprehensive land use or water quality plans. Under these types of plans, the use of on-site systems would be restricted by land use or water quality goals. However, the indirect control also includes regulation of systems by restricting or completely prohibiting their use in certain areas. Thus, comprehensive planning could result in a reduction of the pressure upon the regulatory program by restricting or excluding on-site systems due to land use or water quality criteria.

Regulatory Techniques

By necessity, any regulatory program must contain certain administrative and enforcement techniques. The purpose of administrative mechanisms is to aid in efficient and thorough regulation of on-site sewage systems within the jurisdiction of the administrative agency. Further, to assure that the regulatory programs are successful in controlling on-site sewerage, it is necessary to include enforcement mechanisms in the program. Often the distinction between these two types of techniques becomes blurred and somewhat arbitrary.

There are only a limited number of generic regulatory techniques available for use in any type of control program. Thus, existing and recommended alternative on-system regulatory programs are constrained to be selected from the same "laundry list" of available regulatory techniques. Most of these techniques may be included in one of the following:

1. Direct controls over the on-site system itself;
2. Controls upon the actors (i.e., designers, installers, owners, etc.);
3. General or indirect controls; and
4. Unfair or unlawful controls.

It must be noted, however, that the regulating agency must have the authority to impose these controls. This authority might be in the form of statutory enabling legislation enacted by the state legislature granting the regulatory agency (at either the state or local level of government) the necessary power to implement the regulatory program. This legislation may or may not be obligatory, that is the delegated agencies may decline to regulate on-site sewerage. Further, the legislation might specifically designate the exact regulatory techniques which are to be used and in some cases might even prescribe the procedure to be used and establish a fee structure.

Alternatively, certain types of governmental agencies possess adequate authority either via the state's constitution or by their general grant of statutory authority (i.e., "police power") to implement a regulatory program consisting of many, if not most, of the available regulatory techniques. One example of such an agency might be the state agency responsible for protecting public health and/or water quality. As a second example, incorporated communities in many states, i.e., cities and villages, have the authority to impose the controls needed for an adequate regulatory program and the courts will likely hold this to be a valid exercise of their police power (See Louisville v. Thompson, (Ct. App. Ky.) 339 S.W. 2d 869 (1960); Early Estates, Inc. v. Housing Board of Review, 93 R.I. 277, 1974 A. 2d 117 (1961); City of St. Louis v. Nash (S. Ct. Mo.) 260 S.W. 985 (1924)). In many states, these "home rule" powers are also available to other units of local government, such as towns and counties.

Obviously then, it is necessary to first determine what unit of government is attempting to actually regulate on-site sewerage and then to carefully assess what authority it has. To be absolutely correct, this assessment must look at statutory and case law, as well as the State Constitution. Often the assessment is made more difficult because more than one unit of government is involved in the program, i.e., a state-county program.

Direct Controls--

Direct controls may be thought of as those techniques which the regulatory agency imposes directly upon the on-site systems, itself.

Permit--A permit is a written warrant granted by a governmental unit or agency which conveys the right to conduct a specific activity usually at an identified location and normally for a given fixed period of time, i.e., for installation of on-site systems, construction must commence typically within one or two years or the permit becomes void. The legislation, ordinance or regulation which establishes the permit process also should impose an additional requirement making it unlawful to conduct the regulated activity without first obtaining a validly issued permit. This permit technique can be used to administer and enforce technical and performance standards, simply by making compliance with these standards a necessary prerequisite for issuance of a permit. Clearly most permit processes places the burden of obtaining and supplying all necessary data and information upon the permit applicant, and often require the applicant to pay a fee as precondition to permit issuance.

Permit issuance is a technique which is often used to regulate on-site sewerage. Generally, the program requires that a permit must be obtained prior to commencing installation of any on-site system. The permit requirements may be varied for different types of on-site systems. In addition, some programs require "use" or "occupancy" permits prior to occupancy of the residence served by the on-site system. It has been proposed that a permit program also be employed to assure adequate and timely maintenance of the system (Stewart, 1977). For example, in Wisconsin a county permit is required for the installation of an on-site system (Wis. Admin. Code H62.20). This county permit program is similar to the programs in Pennsylvania and Maine (State of Maine Plumbing Code Sec. 2.2, Dept. of Health and Welfare; Penn. Sewage Facilities Act, P.L. 1535 as amended by 35 P.S. 750). The use of permits either at the local or state government level is desirable because it presents the regulatory agency with notice of all proposed system installations. This is also an inexpensive means of providing the agency with data about the system. Such data is available for compliance checks of the system during construction. Also included under the rubric of this technique are conditional permits, defined as one which is valid only until the occurrence of an event or the failure to comply with a requirement. The placard, stop work order or "red tag" might be used to show this occurrence or failure.

Plan review and approval--A review of project specifications and drawings is a second direct control which may be required prior to undertaking specified activities. Control is assured since the review process can be used to make sure that the regulatory standards will be implemented as part of the activity.

Plan review and approval are frequently included in on-site sewerage regulatory programs. For example, Wisconsin does not require the submission of plans for state approval of conventional on-site systems serving single families; however, pursuant to a recent amendment, the state now requires county review of such plans (Wis. Admin. Code H62.20).

Installation inspection--Inspection is a third technique which is used to assure that other control requirements are met. The types of inspections which have been used range from the inspection of the proposed site prior to its approval for installation of an on-site system, to compliance inspections made after the system is completely installed. Included within this range are one or more inspections during construction, e.g., the "pre-coverup" inspection would be included here. The inspection technique can be included as part of the permit or licensing process whereby the permittee is required to give notice to the regulatory agency at specified periods in the construction of an on-site system. Of course, the application for the permit clearly puts the regulatory agency on notice and as a condition of permit issuance, the applicant may be deemed to have given his consent to a pre-issuance inspection, as well (See v. City of Seattle, 387 U.S. 541, 546 (1967)). In addition, inspections may be performed by the regulatory agency upon a random basis or upon receipt of a complaint. For example, Dane County, Wisconsin now makes a minimum of two inspections for each septic tank system installed: one at the proposed site prior to issuance of a permit and the other during construction. Other inspections may be made when deemed necessary. The county regulatory agency charges a fee for the permit to cover the cost of these inspections.

Access to the property for the purpose of making inspections has been the subject of several court cases. As a result of these cases, the United States Supreme Court has held that under certain circumstances, inspections of property might be a "search" within the meaning of the fourth and fourteenth amendments of the U.S. Constitution. Unless consented to, such an inspection could only be conducted or compelled under a search warrant. [see Camara v. Municipal Court of San Francisco, 387 U.S. 523 (1967) and companion case See v. City of Seattle, 387 U.S. 541 (1967).] . Clearly, under the U.S. Constitution a nonconsentual inspection of a residence would require a search warrant. The courts have also extended this fourth amendment protection to include out-buildings and surrounding land (English law recognized this land as the curtilage or court-yard area). Thus, it is likely that a warrant might be needed if permission cannot be obtained. For non-criminal proceedings, many state statutes now prescribe the procedure for the issuance of an administrative warrant, since the standard search warrant procedure is generally not available. For an example of such a search warrant see: Wisconsin Statutes, 1975, Section 66.123 for specific wording and Section 66.122 for the procedure which must be used to obtain such a warrant.

Maintenance assurance and monitoring--Maintenance assurance and monitoring requirements are direct control techniques similar to the installation inspection techniques. These techniques can be used to assess the compliance with the regulatory program and to assess the success of enforcement of various types of other regulatory techniques. These maintenance and monitoring techniques might involve sanitary surveys, chemical or dye tests and aerial photography to determine the effectiveness of other control techniques as well as water quality monitoring. The same constitutional constraints would apply to nonconsentual access to property as was discussed for inspections in general. For example, the Auburn Lake Trails Subdivision in Georgetown Divide Public Utility District, El Dorado County, California, undertakes a water quality monitoring program to determine whether any of the septic tank systems are effecting the surface water quality (Georgetown Divide Public Utility District, 1972). In addition, the El Dorado Irrigation District has provided maintenance assurance programs to several subdivisions within the district (Winneberger and Anderman, 1972). Their program involves both annual physical inspection and maintenance of septic tank systems within the district.

Bonding or other surety--The technique of requiring the posting of a bond or other surety as a guarantee that the requirements of the regulatory program will be complied with must be mentioned as a direct regulatory control technique. Typically, the enabling legislation, ordinance or regulations contains either a schedule listing the amount of the bond or surety or it provides a formula, from which the regulatory agent can calculate the amount. Usually the bond or surety is a condition precedent to issuance of a permit. It is generally returned after a certain time, e.g., 6 months to 1 year after occupancy of the dwelling and use of the system.

This technique is not without its shortcomings. In particular, while a bond might be posted to assure the proper installation and/or function of an on-site system, the regulatory agent might be reluctant to attempt to collect on the bond in the event of improper installation and/or failure of the system. The nature of the bond always involves a question of fact, i.e., was the installation improper, did the system fail to function, and why. Such questions of fact invite court actions and, thus, posting a bond or requiring a surety does not always bring about the regulatory result desired by the municipality or state agency.

Controls Upon Actors--

On-site regulatory programs may use techniques other than direct controls, as discussed above, to obtain the primary objective of public health protection. The most important method of so doing is to regulate those who act on the systems. The actors most likely to be regulated are the soil testers, designers and installers of on-site systems, as well as those who service or maintain the systems, i.e., liquid waste pumpers/haulers.

Licensing--Licensure of qualified individuals has long been recognized as a legitimate function of the state under its police power. The United States Supreme Court described this power as follows:

> "The power of the state to provide for the general welfare of its people authorizes it to . . . secure them against the consequences of ignorance and incapacity, as well as deception and fraud. As one means to this end it has been the practice of different states, from time immemorial to exact in many pursuits a certain degree of skill and learning upon which the community may confidentially rely, their possession being generally ascertained upon an examination of parties by competent persons, or inferred from a certificate to them in the form of a diploma or license from an institution." Dent v. West Virginia, 129 U.S. 114, 122 (1889).

As stated by the Supreme Court, the requisite degree of skill may be ascertained by an examination or inferred from a diploma or license from an institution.

Some of these regulatory programs have required that the designers of the on-site systems be licensed professional engineers, architects or plumbers. Additionally, some programs limit those who may actually install the systems to those who are licensed plumbers, architects, septic system installers, etc. For example, in Illinois the contractors who install on-site systems pay an annual fee of 50 dollars and are licensed by the state (State of Illinois, Private Sewage Disposal Licensing Act). In Wisconsin, state law prohibits anyone other than a licensed master plumber from installing on-site systems (Wis. Stat. 1975, Sec. 145.06).

However, it is important to realize that the regulatory program is not constrained to rely on pre-existing licensing programs, but may, in fact, provide a training and/or examination program and establish its own licensing program. For example, Wisconsin recently created a program to license those who perform the soils evaluations for the suitability of sites for on-site soil disposal systems (Wis. Admin. Code H62.20, H64). Similarly, several agencies have apparently incorporated the licensure of the liquid waste haulers/pumpers into their regulatory programs. The State of Delaware annually licenses liquid waste pumpers (Del. Water Poll. Contr. Reg. No. 12) as do the states of Florida (State of Fla. Dept of Poll. Contr. Rules, Chap. 17-13), Illinois (State of Ill. Private Sewage Disposal Licensing Act) and Wisconsin (Wis. Admin. Code, Chap NR 113 and Wis. Stat. 1975, Sec. 146.20). The Commonwealth of Pennsylvania has a certification program similar to licensing for sewage enforcement officials because each municipality in Pennsylvania is required to employ such an official (Penn. Sewage Facilities Act P.L. 1335 as amended by 35 P.S. 75 and Dept. of Envir. Res. Rules and Reg., Chap. 71). Also many states license sanitarians.

Registration--Registration requirements are sometimes used as a regulatory technique. Generally, the difference between this and licensure is that registration is often only a bookkeeping, non-discretionary listing of those who are performing certain functions. That is, registration might be nothing more than the keeping of an updated list of all those who have applied to the agency or otherwise indicated an interest in performing these functions.

However, there appears to be a shift in preference from licensing to registration based upon the fact that licensing is only as good as its follow-up. Some licensing boards have tended to become co-opted by the licensees, and, even if not so controlled the regulating boards generally have been quite reluctant to revoke a license. Therefore, licensing often gives a false sense of security to the consumer. Thus, there is a movement toward regulation and away from the previously preferred policy of licensing.

One spinoff technique available to those agencies which impose licensure requirements upon some or all of those who perform actions related to on-site systems is to limit the issuance of permits solely to those who are properly licensed or registered.

General or Indirect Controls--

General or indirect controls are those which the regulatory agency seldom has the ability to influence or determine completely. Examples of these controls are zoning and land use policies. While the regulatory agency might have the enforcement or administrative responsibilities for zoning or other land use techniques, it is unlikely that the agency itself can adopt zoning land use ordinances. One exception to this would be the denial of the issuance of building permits until all on-site requirements have been complied with. It is possible that the same regulatory agency would process both permits. Along these lines, some local governmental units have in place, highly structured interlocked procedures for the review of development proposals. Thus, this procedure acts as an indirect control upon on-site systems included in any development proposal.

A second example of an indirect control would be the existence of public policies in favor of or against certain regulated actions. These policies might aid or hinder the regulatory agency in the administration of its program, e.g., the "aggressiveness" and priority placed upon enforcement; budget and authorization for the agency will influence the number and professionalism of the administrative staff, as well as the quality of their program.

Unfair or Unlawful Controls--

The regulatory agency might seek to control certain portions of its on-site system regulatory program by establishing excessive fee requirements, by delay in processing applications or by unjustified denial of permits. These techniques are not desirable control techniques and are just mentioned to point out that they do exist and have been used in the past.

Summary--

In conclusion, it can be seen that a typical regulatory program to control on-site wastewater treatment and disposal systems could consist of direct controls such as permit issuance, inspection, plan review and possibly bonding, and/or maintenance assurance and monitoring. Note, that the program could and probably should, involve most of these direct control techniques. In addition, the program should preferably require licensing or at least regulation of system designers, installers and pumpers as well as the persons enforcing the program. The types of regulations would principally involve specification standards and possibly some performance standards especially for surface discharge units. Thus, while each regulatory program would be unique, each would contain many of the same regulatory techniques and standards.

Theory of the Regulation of On-Site Systems

The previous section gave a brief introductory examination of the types of regulations and the regulatory techniques which are contained in regulatory programs. This section examines the theory of these regulatory programs by first examining the three regulatory phases of on-site systems and concludes by briefly exploring the wide spectrum of types of regulatory programs used to control on-site systems.

Three Regulatory Phases of On-Site Systems--

The three phases where regulation of any on-site system is needed are the initial installation phase, the operational phase and the failure phase.

However, before an examination of these phases of a good regulatory program can be meaningful, it is necessary to briefly examine the problems which arise or have been attributed to the use and misuse of on-site systems. A good regulatory program is one which addresses these problems and adequately deals with them.

Problems--The threat to public health due to water-borne diseases is generally the major problem raised when discussing on-site sewerage. Typically, those individuals who supply their own water have the highest risk of water-borne disease. In the United States, from 1961 to 1970 there were a total of 128 known outbreaks of water-borne disease (defined as at least two reported cases) attributed to drinking water which caused about 46,000 illnesses and 20 deaths (Craun and Mc Cabe, 1973). Ninety-four of these outbreaks occurred in private water supplies and the majority of these outbreaks were classified as gastroenteritis. This same source compiled outbreak data for the 25 year period of 1946 to 1970 and concluded that 71 percent of the outbreaks (of a total of 358) occurred in private water supplies. It is important to note that many outbreaks of water-borne illness go unreported, so that the true incidence of disease may be assumed to be much higher.

Disease outbreaks could be drastically reduced by eliminating the travel of pathogens into water supplies. It has been argued that improper siting and design of the on-site system in the initial installation phase and failing systems at the end of their life cycle are the major sources of contamination. For this reason, these systems pose a potential threat to public health. Thus, many health officials have adopted the attitude that the use of on-site systems is to be generally discouraged -- seeking replacement where possible with central systems.

Another potential problem associated with on-site systems is their inability to remove potentially troublesome chemicals found in the wastewaters, typically nitrogen and phosphorus. These chemicals represent both a potential public health threat (i.e., nitrates causing infant methemoglobinemia) and a source of undesirable nutrients affecting both surface and groundwaters. There are two stages in the life cycle of on-site systems where this problem may occur. The first is due to improper siting and design of the system. The second is the end of the life cycle of the system when it has failed. Either improper siting or a failed system may result in contamination of surface or groundwaters with unwanted chemicals. Neither the public health aspects nor the contamination of surface or groundwaters will be discussed in detail here; however, it should be noted that since 1945 about 2,000 cases, including fatal poisonings, of methemoglobinemia have been reported worldwide (Shuval, 1970). Further note that there are many other chemicals which might occur in wastewaters which are not discussed here.

Most on-site systems have the attendant problem of limiting development. On-site systems which rely on soil for final disposal function properly only if located on suitable sites. In many places in the United States, suitable sites for on-site soil disposal are not available. In those jurisdictions which have a good administrative program of limiting installations to only suited sites, the resulting limited development has been referred to by some as de facto zoning. That is, the siting requirements necessary for soil disposal tend to limit the amount of land available for development. In the past, many jurisdictions have relied on these requirements to provide them with a means of land use control. As innovative systems, which do not have as stringent siting requirements or do not rely on the soil for disposal become more widely accepted, this technique of land use control will be lost.

This problem of limiting development arises only at the initial or first phase in the life cycle of an on-site system. However, the magnitude of this problem is quite large. One source estimated that about 68 percent of the United States is unsuited for the conventional soil disposal system (Wenk, 1971). Thus, the potential for a problem of limiting development is great, especially as more and more jurisdictions improve their programs of limiting installations to only suited sites.

Economic and financial hardship problems often arise when on-site systems are employed. Again, these problems generally occur during the initial phase in the life of an on-site system; however, these same type problems may occur in the failure or final phase in the life cycle. A typical example of this problem arises when the homeowner, after purchasing his site, discovers that it is unsuited for an on-site system. Several things may occur, first, the value of his lot and home, if already built, is greatly diminished. Second, he may plead financial hardship to the regulatory authorities in an effort to receive approval to install the system despite the lack of suited soil. This same type problem may also occur when an existing homeowner's system fails. He, of course, has a much stronger financial hardship argument to raise if he is unable to find suitable soil on his lot; because very few regulatory authorities will ever require a homeowner to vacate his home. Of course, if the homeowner prevails, the installation of systems on unsuited sites may cause the public health and chemical problems as discussed.

Regulatory phases--The initial installation phase consists of proper siting, design and construction of the on-site system. Through proper controls the potential danger to public health and pollution of surface and ground waters may be avoided, as well as controlling economic hardship problems.

It is during this first phase that the regulatory program can be most cost effective in minimizing public health and water pollution problems. Regulatory emphasis must be given to assuring that on-site systems are installed only on suitable sites, but assurance of proper design and construction techniques are necessary as well.

While the regulation of this first phase is ideally suited to avoid health and water quality problems, the program also can avoid possible economic hardships. This is possible because landowners are better served if the regulatory program prevents them from installing on-site systems on unsuitable sites. Thus, even though the owner of undeveloped land might argue financial and economic hardships if he cannot develop the land, the hardships may be minimal when compared to the hardships which would result if the on-site system were permitted and later failed.

The second regulatory phase is concerned with operation and maintenance. The problems of public health and pollution of the surface and groundwater may occur if there is inadequate control over proper operation and maintenance. While there are very few operational or maintenance requirements for a septic system, some of the more innovative on-site treatment and disposal systems have more extensive requirements. Whether the system's operation and maintenance requirements are straightforward or elaborate, a good regulatory program should impose controls at this second phase in the life cycle.

The third phase occurs when a system fails. This phase involves both the detection of failure and the imposition of the necessary corrective actions. Detection may result from an active role taken by the regulatory agency such as: 1) a systematic, scheduled inspection of every on-site system in its jurisdiction, 2) random inspections of systems, or 3) a sanitary survey of a given region or area for purposes of both locating systems and determining whether they are functioning properly. In addition, failing systems may be brought to the attention of the regulatory agency by citizen complaints or self-reporting by owners of failing systems.

Despite the manner of detection, an adequate regulatory program must be empowered to take necessary action once the failing systems are noted. At a minimum, the regulatory agency must have the authority to order repair, replacement or abandonment of failing systems.

This is the most difficult phase to regulate. However, the problems of public health, water pollution and economic hardships may be attenuated or avoided by proper regulatory control.

Spectrum of Regulatory Programs--

The regulatory program to control on-site systems varies widely from state to state and is probably subject to at least an equal amount of variation among local regulatory authorities within each state. Existing regulatory programs vary from total regulation of all on-site systems to almost no regulation whatsoever. Within this spectrum, are several intermediate programs which split responsibilities for setting standards, inspection, permit issuance and enforcement provisions between the state and local authorities.

Despite this wide range of state programs, it is possible to categorize them into four general types (Patterson, et al., 1971): 1) complete state regulation of all on-site systems, 2) split of regulatory control between state and local agencies, 3) delegation by the state to local governmental units, and 4) virtually no state action (not even delegation to local authorities).

Some of the states with complete state level regulatory programs include (Plews, 1977):

Connecticut
Delaware
Hawaii
New Hampshire
Oregon
Rhode Island

These states typically require a permit for on-site system installation and require an inspection by the state agency (Patterson, et al., 1971).

This type of regulatory program is generally considered to be the most effective because the pressures to weaken on-site regulatory programs are not usually as effective at the state level as at the local level. In addition, since states typically have or can obtain greater expertise and technical knowledge than most local units of government, state agencies should assist or perform many of the regulatory control techniques.

States which divide the regulatory control between state and local authorities include (Plews, 1977):

Alaska
California
Colorado
Florida
Maine
Montana
Nevada
New Jersey
New Mexico
Oklahoma
Pennsylvania
South Carolina
South Dakota
Texas
Utah
Vermont
West Virginia

Specifically, Pennsylvania has a strong state program, but it requires that each municipality employ at least one state trained official to implement the program at the local level ("Penn. Sewerage Facilities Act" P.L. 1535 as amended by 35 P.S. 750 and Dept. Environ. Res. Rules and Reg., Chap. 71). Similarly, Maine requires a local plumbing inspector to issue permits for residential systems (Maine State Plumbing Code, Part II, Sec. 2.2).

States which have delegated regulatory responsibility to the local governmental or health authorities include:

Alabama
Arizona
Georgia
Indiana
Iowa
Kentucky
Louisianna
Michigan
New York
North Carolina
Ohio
Tennessee
Virginia
Washington
Wisconsin

These states have been identified as deferring functions such as the permit and inspection responsibilities to local regulatory agencies (Plews, 1977). Some states, such as Ohio, have adopted a state code of minimum standards and specifications for on-site systems (Ohio Health Dept. H.E. 20). In states having adopted minimum standards, the local health authorities' codes and standards generally must be as stringent as the state codes.

States which have been identified as having virtually no state regulatory programs include (Plews, 1977):

Missouri
Nebraska
North Dakota
Wyoming

Wyoming apparently only has a limited advisory educational program in which the state attempts to provide assistance to the general public regarding on-site systems. North Dakota only takes enforcement against owners of systems which result in a water pollution or public health problems (Patterson, et al., 1971).

Analysis of Current State Regulatory Programs

While the range of regulatory programs in the 50 states is quite broad, the on-site system standards and specifications imposed by the states vary to an even greater degree. Many of the states use the Manual of Septic Tank Practice (USPHS, 1967) as the basis for their standards. However, many states and localities have diverged significantly from this basic standard (Plews, 1977).

This large variation in the standards and specifications has been documented by Plews (1977). He analyzed the state codes, regulations and guidelines governing on-site sewerage in the following states:

Alabama
Alaska
Arizona
California
Colorado
Connecticut
Delaware
Florida
Georgia
Hawaii
Idaho
Illinois
Indiana
Iowa
Kentucky
Louisianna
Maine
Michigan
Missouri
Montana
Nebraska
Nevada
New Hampshire
New Jersey
New Mexico
New York
North Carolina
North Dakota
Ohio
Oklahoma
Oregon
Pennsylvania
Rhode Island
South Carolina
South Dakota
Tennessee
Texas
Utah
Vermont
Virginia
Washington
West Virginia
Wisconsin
Wyoming

Plews concluded that:

> "... the Manual of Septic Tank Practice has had some influence. However, the diversity in certain requirements is questionable and obviously many of the documents have been developed through political compromise rather than by sound technical advice."

Tables D-1, D-2, D-3, D-4 and D-5 show the wide range of variation in the specification standards currently used in the various states to regulate: 1) septic tank capacity, 2) absorption field setback distances, 3) absorption field percolation restrictions and sizing methods, 4) soil depth and surface discharge restrictions, and 5) absorption field design requirements, respectively.

Licensing--

The licensure of those who interact with on-site systems is a valid regulatory technique which might be used to control both conventional and non-conventional on-site systems. Several programs exist which currently license those individuals who install, inspect, operate and/or service on-site systems. The use of licensure programs can be an efficient method to assure control over the individual who interacts with the on-site system. However, caution must be exercised. Each licensure program must be evaluated individually before incorporating it as a regulatory technique because such programs may suffer from lack of policing of the licensee and excessive use of "grandfather" privileges.

TABLE D-1. SEPTIC TANK DESIGN STANDARDS USED IN THE U.S. (Plews, 1977).

	Septic Tank Capacity in Gallons By Number of Bedrooms				
States	1	2	3	4	5
Alabama	1000	1000	1000	1200	1400
Alaska	750	750	900	1000	1250
Arizona	960	960	960	1200	1500
Arkansas					
California					
Colorado	750	750	900	1000	1250
Connecticut	1000	1000	1000	1250	1500
Delaware	750	750	750	1000	1250
Florida	750	750	900	1000	1200
Georgia	750	750	900	1000	1250
Hawaii	750	750	1000	1200	1350
Idaho	750	750	900	1000	1250
Illinois					
Indiana	750	750	900	1100	1250
Iowa	750	750	1000	1250	1500
Kansas					
Kentucky	750	750	900	1000	1250
Louisiana	500	750	900	1150	1400
Maine	750	750	900	1000	1250
Maryland					
Massachusetts					
Michigan					
Minnesota					
Mississippi					
Missouri					
Montana	750	750	900	1000	1250
Nebraska	750	750	900	1000	1250
Nevada	1000	1000	1000	1000	1250
New Hampshire	750	750	900	1000	1250
New Jersey	750	750	900	1000	1250
New Mexico	750	750	900	1000	1250
New York	750	750	900	1000	1250
North Carolina	750	750	900	1000	1250
North Dakota					
Ohio	1000	1000	1500	2000	2000
Oklahoma	1000	1000	1000	1000	1250
Oregon	750	750	900	1000	1250
Pennsylvania	900	900	900	1000	1100
Rhode Island	750	750	900	1000	1250
South Carolina	890	890	890	?	?
South Dakota	1000	1000	1000	1250	1500
Tennessee	750	750	900	1000	1250
Texas	750	750	1000	1250	1500
Utah	750	750	900	1000	1250
Vermont	1000	1000	1000	1000	1500

(continued)

TABLE D-1 (continued)

	Septic Tank Capacity in Gallons By Number of Bedrooms				
States	1	2	3	4	5
Virginia	30 Hour Detention - 100 Gallons Per Person				
Washington	750	750	900	1000	1250
West Virginia	750	750	900	1000	1250
Wisconsin	750	750	975	1200	1375
Wyoming	750	750	900	1000	1250

TABLE D-2. ABSORPTION FIELD DESIGN STANDARDS USED IN THE U.S. (Plews, 1977)

States	Setback Distance Drainfield To Well In Feet	Setback Distance Drainfield To Surface Water In Feet
Alabama	50-75	?
Alaska	50-100	50-100
Arizona	50-100	100
Arkansas		
California		
Colorado	100	50
Connecticut	75	50
Delaware	50-100	50
Florida	75-100	50
Georgia	100	50
Hawaii	50	50
Idaho	100	100-300
Illinois		
Indiana	50-100	50
Iowa	100-200	25
Kansas		
Kentucky		
Louisiana	100	?
Maine	100-300	50-100
Maryland		
Massachusetts		
Michigan		
Minnesota		
Mississippi		
Missouri		
Montana	100	100
Nebraska	100	50
Nevada	100	100
New Hampshire	75	75
New Jersey	50-100	50
New Mexico	100	50
New York	100	100
North Carolina	100	50
North Dakota		
Ohio	50	?
Oklahoma	50-100	50
Oregon	50-100	50-100
Pennsylvania	100	50
Rhode Island	100	50
South Carolina	100	50
South Dakota	100	100
Tennessee	50	25
Texas	100-150	75
Utah	100	100

(continued)

TABLE D-2 (continued)

States	Setback Distance Drainfield To Well In Feet	Setback Distance Drainfield To Surface Water In Feet
Vermont	100	50
Virginia	35-100	50-100
Washington	75-100	100
West Virginia	100	100
Wisconsin	50-100	50
Wyoming	100	50

TABLE D-3. ABSORPTION FIELD DESIGN STANDARDS USED IN THE U.S. (Plews, 1977)

States	Minimum Percolation Restriction	Sizing Methods
Alabama	None	Perc
Alaska	None	Perc & Soils
Arizona	None	Perc
Arkansas		
California		
Colorado	None	Perc
Connecticut	None	Perc
Delaware	Yes	Perc
Florida	None	Perc & Soils
Georgia	None	Perc & Soils
Hawaii	None	Perc
Idaho	None	Perc & Soils
Illinois		
Indiana	None	Perc
Iowa	None	Perc & Soils
Kansas		
Kentucky	?	Perc
Louisiana	None	Perc
Maine	None	Soils
Maryland		
Massachusetts		
Michigan		
Minnesota		
Mississippi		
Missouri		
Montana	Yes	Perc & Soils
Nebraska	No	Perc
Nevada	Yes	Perc
New Hampshire	None	Perc
New Jersey	Yes	Perc & Soils
New Mexico	Yes	Perc & Soils
New York	None	Perc & Soils
North Carolina	None	Perc & Soils
North Dakota		
Ohio	None	Soils
Oklahoma	None	Perc Test
Oregon	None	Soils
Pennsylvania	Yes	Perc
Rhode Island	None	Perc
South Carolina	None	Perc & Soils
South Dakota	Yes	Perc
Tennessee	None	Perc & Soils
Texas	Yes	Perc & Soils
Utah	None	Perc
Vermont	None	Perc & Soils

(continued)

TABLE D-3 (continued)

States	Minimum Percolation Restriction	Sizing Methods
Virginia	None	Perc & Soils
Washington	Yes	Perc & Soils
West Virginia	None	Perc
Wisconsin	None	Perc & Soils
Wyoming	None	Perc

TABLE D-4. SPECIAL SITE RESTRICTIONS MADE IN THE U.S. (Plews, 1977)

States	Required Soil Depth Below Bottom Of Trench In Feet	Allows Surface Discharge
Alabama	4	?
Alaska	4	No
Arizona	4	No
Arkansas		
California		
Colorado	No Minimum	Yes
Connecticut	1.5	No
Delaware		
Florida	1.5	No
Georgia	No Minimum	No
Hawaii	No Minimum	Yes, Conditional
Idaho	4	No
Illinois		
Indiana	?	No
Iowa	1.5[a]	Yes
Kansas		
Kentucky	?	No
Louisiana	None	Yes
Maine	2	Yes
Maryland		
Massachusetts		
Michigan		
Minnesota		
Mississippi		
Missouri		
Montana	4	No
Nebraska[b]	?	No
Nevada	4	No
New Hampshire	4[a]	No
New Jersey	4	No
New Mexico		
New York	2	No
North Carolina	1	Yes
North Dakota		
Ohio	4[a]	Yes
Oklahoma	4	No
Oregon	1.5[a]	No
Pennsylvania	4	No
Rhode Island	3	No
South Carolina	6"	No
South Dakota	4	No
Tennessee	4[a]	No
Texas	4	
Utah	1	No

(continued)

TABLE D-4 (continued)

States	Required Soil Depth Below Bottom Of Trench In Feet	Allows Surface Discharge
Vermont	4	No
Virginia	No Minimum	Yes
Washington	3[a]	No
West Virginia	4	No
Wisconsin	3[a]	No
Wyoming	4	Yes

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[a] Allows less with special design

[b] Guidelines

TABLE D-5. ABSORPTION FIELD DESIGN REQUIREMENTS AND SIZING METHODS USED IN THE U.S. (Plews, 1977)

States	Minimum Spacing Between Lines In Feet	Minimum Soil Cover Over Trench In Inches	Range of Drainfield Widths in Inches
Alabama	6	6	18-36
Alaska	6	12	12-36
Arizona	6	12	12-18
Arkansas			
California			
Colorado	6	12	18-36
Connecticut	6-9	6	18-36
Delaware	6.5-7.5	9	12-36
Florida	6-8	12	18-24
Georgia	10	12	18-36
Hawaii	6	12	18-36
Idaho	6	12	12-36
Illinois			
Indiana	6-7.5	12	18-36
Iowa	7.5	12	18
Kansas			
Kentucky	?	None	?
Louisiana	?	6-12	12-18
Maine	10	2-6	24
Maryland			
Massachusetts			
Michigan			
Minnesota			
Mississippi			
Missouri			
Montana	6	12	12-36
Nebraska	6	6	18-36
Nevada	6	4-6	12-24
New Hampshire	6-7.5	6	12-36
New Jersey	6-7.5	12	18-36
New Mexico			
New York	6	12	24
North Carolina	8	12	18-36
North Dakota			
Ohio	6	6	8-30
Oklahoma	8	10	24
Oregon	10	6	24
Pennsylvania	6	12	12-36
Rhode Island	6	12	18
South Carolina	10	9	18-36
South Dakota	6	?	?
Tennessee	6	12	18-36
Texas	7	6	18-36
Utah	6-7.5	12	12-36
Vermont	6	6	12-48
Virginia	6-9	None	18-36

(continued)

TABLE D-5 (continued)

States	Minimum Spacing In Feet Between Lines In Feet	Minimum Soil Cover Over Trench In Inches	Range of Drainfield Width In Inches
Washington	6	6	18-36
West Virginia	6	12	12-36
Wisconsin	10	12	18-36
Wyoming	6-7.5	6-12	12-36

Table D-6 lists the states which register sanitarians (Winston, 1975). Registered sanitarians, it is assumed, have the requisite knowledge and skills to adequately deal with the public health aspects of on-site systems. Thus, it is possibly this official who is chosen to administer on-site regulatory programs. Of course, this assumption varies from state to state depending upon the quality of the licensure program.

While Registered Sanitarians may be qualified to administer a regulatory program for on-site systems, at least one state has perceived the need for a more specialized individual. Pennsylvania adopted a program to certify sewage enforcement officials to directly administer the bulk of its on-site regulatory program ("Penn. Sewage Facilities Act" P.L. 1535 as amended by 35 P.S. 750).

Table D-7 lists those states which have either a mandatory or voluntary certification program for wastewater treatment plant operators. This is germain because a certified treatment plant operator may be required to operate alternative treatment systems which are more complex than the conventional septic tank system. For example, in Wisconsin, a licensed treatment plant operator is required to operate all treatment plants in the state. Many on-site systems (other than septic tank systems which are excluded by specific wording in the rule) are defined by Wisconsin as "treatment plants" and, therefore, would require a licensed operator (Wis. Admin. Code Chap NR 114).

Suggested Improvements in On-Site System Regulatory Programs

Several recommendations can be made to improve regulation of on-site system. However, due to the variation in state regulatory programs and different state constitutional limitations and requirements, some of these recommendations may not be applicable or possible in all states. Where applicable, enactment of enabling legislation may be required.

These suggestions are discussed under the headings of the three regulatory phases of an on-site system discussed previously. In some cases, incorporating a suggested improvement in one phase may bring about improvements in another. Such suggestions are discussed in the phase where the most improvement might be effected.

Initial Installation--

State Permit Program--Many local regulatory officials are subjected to political pressure to approve the installation of systems on unsuited sites. Also, some local authorities have reported that their boards of appeal are subject to similar pressures and consequently often override denials made by the local authority. In a survey made of 31 county regulatory officials in Wisconsin, 24 expressed the need for increased job security indicating the existence of political pressure (Stewart, 1974).

A state permit program is a method of avoiding undue pressure to approve system installations on unsuited sites. The chance for direct political pressure should be considerably less at the state level. Additional advantages

TABLE D-6. STATES WHICH REGISTER SANITARIANS (Winston, 1975)

Alabama
Arkansas
California
Colorado
Connecticut
Florida
Georgia
Hawaii
Idaho
Illinois
Indiana
Kentucky
Louisiana
Maryland
Massachusetts
Michigan
Mississippi
Montana
Nebraska
Nevada
New Jersey
New Mexico
North Carolina
Oklahoma
Oregon
Rhode Island
South Carolina
South Dakota
Tennessee
Texas
Utah
Virginia
Washington
West Virginia
Wisconsin

TABLE D-7. CERTIFICATION PROGRAMS-BY STATE (DASHES INDICATE NO INFORMATION AVAILABLE) (Commission on Rural Water, 1974)

State	Mandatory	Voluntary	State	Mandatory	Voluntary
Alabama	X		Montana	X	
Alaska	-	-	Nebraska		X
Arizona	X		Nevada		X
Arkansas	X		New Hampshire	-	-
California	X		New Jersey	X	
Colorado		X	New Mexico		X
Connecticut	X		New York	X	
Delaware	X		North Carolina	X	
Florida	X		North Dakota	X	
Georgia	X		Ohio	X	
Hawaii		X	Oklahoma	X	
Idaho		X	Oregon		X
Illinois	X		Pennsylvania	X	
Indiana	X		Rhode Island		X
Iowa	X		South Carolina	X	
Kansas		X	South Dakota	X	
Kentucky	X		Tennessee	X	
Louisiana	X		Texas	X	
Maine	X		Utah		X
Maryland	X		Vermont	X	
Massachusetts	X		Virginia	X	
Michigan	X		Washington		X
Minnesota	X		West Virginia	X	
Mississippi	-	-	Wisconsin	X	
Missouri		X	Wyoming	-	-

arising from such a program are: 1) uniformity of regulations throughout the state, 2) possible increase of available resources for commitment to the regulatory program, and 3) increased number of experienced personnel, or at least an increase in the ability of the state to retain such personnel experienced in soils and on-site system regulation.

While these advantages exist, disadvantages have been put forth by state regulatory program detractors. Principally, these include: 1) failure at a state permit program to take into consideration local variables such as soil conditions, etc., and 2) loss of responsiveness to local needs. Both of these should not be real disadvantages since the basis for on-site regulation is the protection of public health regardless of the regulatory level.

The use of a permit represents the basis of almost all regulatory programs regardless of the administering level of government. Its widespread use arises because of obvious enforcement advantages. The permit application gives the regulatory agency notice of intended system installation. Therefore, if a landowner installs a system without obtaining a permit, the regulatory agency could establish a *prima facia* case simply by establishing in court that 1) under the regulatory program a permit is needed, and 2) the landowner did not have a permit. This makes enforcement much easier because it is not necessary to prove that the system constituted a nuisance or threat to public health.

The previous section listed states having a complete regulatory program at the state level, which likely includes a permit program. For example, the Delaware Department of Natural Resources and Environmental Control issues installation permits (Del. Water Poll. Cont. Reg. #2). Similarly, the Hawaii Department of Health requires landowners to obtain written approval prior to installation of an on-site system (Hawaiian Publ. Health Reg. Chap. 38), and the New Hampshire Water Supply and Pollution Control Committee issues installation permits (N.H. Rev. Stat. Annot. Chap 149-E).

In summary, permits are used as a regulatory technique to control the location of on-site systems and to facilitate enforcement. Further, if the program is administered at the state level, presumably there is less chance for local political pressure to be exerted to obtain permits for unsuited sites. Such a program would not represent any increased costs to the state if the permit fee were set to recover the costs of administration.

Review of local permits--As an alternative to a state permit program, a state agency could provide similar insulation against local political pressures through a program of review of installation permits issued by the local regulatory agencies. This regulatory technique would not require that all permits be reviewed but only some of the permits selected at random. However, it would be necessary to require each local agency to submit a copy of every permit issued to the state. The state agency would then have a limited period of time to act on the permits. Thus, the permit issued by the local agency would: 1) not become effective for a given number of days to allow time for review by the state agency, and 2) be subject to revocation by the state agency.

It is likely that enabling legislation would be required in most states to provide for a state review program. The legislation should mandate that: 1) all local regulatory programs must contain an installation permit requirement, 2) copies of all permits must be forwarded to the reviewing state agency, and 3) the state agency shall review a random selection of all of the permits received within a set time period.

It is not known whether any state currently employs a permit review program. Stewart (1974) proposed statutory language for Wisconsin, which, if adopted, would have required the Department of Health and Social Services to review all local (county) installation permits. The language was as follows:

> SECTION 1. 59.07 (51) of the statutes is amended to read:
> 59.07 (51) Building and Sanitary Codes. Adopt building and sanitary codes, make necessary rules and regulations in relation thereto and provide for enforcement of such codes, rules and regulations by forfeiture or otherwise. Copies of all permits issued pursuant to sanitary codes adopted under this section shall be sent to Department of Health and Social Services and shall be issued subject to the requirements of s. 145.025. Such codes, rules and regulations shall not apply within cities and villages which have adopted ordinances or codes concerning the same subject matter.
>
> SECTION 2. 145.025 of the statutes is created to read:
> 145.025 REVIEW OF COUNTY PERMITS. (1) Counties Shall Forward Copies of Permits. Each county shall within 5 days from the date of issuance, forward a copy of the permit application for each permit issued pursuant to sanitary codes, rules or regulations adopted pursuant to s. 59.07(51).
> (2) Departmental Review. The department, either at its central or district offices, shall review all permits; and from information contained on the permit application, soil surveys, and its own information, the department shall make a finding that the issuance of the permit is or is not in compliance with the plumbing code. The department is empowered to make its own investigation of any facts necessary to make such a finding.
> (3) Department May Overrule. When the department makes a finding that the issuance of the permit does not comply with the state plumbing code, the department shall within 15 days after receipt notify the county which issued the permit that said permit is cancelled. Further, the department shall notify the applicant that the permit has been cancelled and such notice shall inform the applicant of the enforcement and review provisions of this section.
> (4) Enforcement. Upon receipt of notice of cancellation, the applicant may not proceed with the construction, alteration or extension for which the permit was required. The circuit court of any county where a permit has been cancelled shall have jurisdiction to abate the use of a constructed facility and to halt further construction and use by injunctive or other appropriate relief. The district attorney shall bring an action to halt all such construction.

> (5) Review. Any applicant who has a permit cancelled under this section may obtain judicial review as provided in ch. 227.

The costs of this permit review program could be included as part of the permit fee. A portion of the fee collected by the local agency would be forwarded to the state reviewing agency along with the permit.

Minimum standards for local regulatory programs--As a second alternative to the state permit program, a state agency with appropriate regulatory authority could establish minimum standards by which local regulatory programs could be reviewed. As with the programs discussed previously, this program would insulate the permit process from local political pressures.

Enabling legislation probably would be necessary to implement this program also. Stewart (1974) proposed statutory language which would have empowered the Wisconsin Department of Health and Social Services to evaluate each local (county) regulatory program on the basis of installation permits issued. If after its review, the Department found the local agency ineffective, it was empowered to suspend the local agency's authority to issue permits. Thus, the penalty for failure to avoid local political pressure in issuing permits is the loss of authority to administer the program and henceforth permit applications must be made directly to the state. The specific statutory language is as follows:

> SECTION 1. 145.025 of the statutes is created to read:
> 145.025 MINIMUM STANDARDS FOR COUNTY SANITARY PROGRAMS. (1) The department is empowered to require each county to forward a copy of all permit applications for permits issued pursuant to sanitary codes, rules and regulations adopted pursuant to s. 59.07 (51). Further, the department is empowered to require each county to certify which of these permits were issued.
>
> (2) The department shall require the counties to forward all permit applications which were granted for a period of 1 year from the effective date of this section, and the department shall review all of said permit applications.
>
> (3) After this 1 year period, the department shall within 60 days issue findings of fact about the effectiveness of each county's sanitary program. The department shall find a sanitary program ineffective if the review of the sanitary permits in subs. (2) established that permits were issued in violation of the site requirements of the state plumbing code. Further, the department may find that a county sanitary program is ineffective for other reasons.
>
> (4) Based on its findings of fact, the department shall issue an order to each county found to have an ineffective sanitary program, ordering it to cease issuing permits.
>
> (5) No person may construct, alter or extend a private domestic sewage treatment and disposal system in any county found to have an ineffective sanitary program, unless that person has first obtained a permit from the department authorizing that system.
>
> (6) The department may issue the permits in subs. (5) and may set a fee for the issuances of the permits.

(7) Review. Any county found to have an ineffective sanitary program may petition the department for a review, under ch. 227.

(8) The department may at any time rescind its order and allow a county to begin issuing permits. However, the department shall review all county permits issued as provided in subs. (2) and the department shall make a finding of fact as required in subs. (3).

This minimum standards review is currently being used by the Commonwealth of Pennsylvania. The local agency issues the installation permit and the Pennsylvania Department of Environmental Resources is responsible for reviewing the administrative performance of the agency. The Department may order a local agency to modify its permit issuance practices to correct any deficiencies found (Penn. Dept. Environ. Res. Rules and Reg. Chap. 71 and 73).

Uniform citation and complaint--The issuance of citations for sanitary ordinance or code violations can be an important regulatory technique. The citation system is currently used by building inspectors in several major American cities to "ticket" owners of buildings who violate local codes (National Institute of Municipal Law Offices, Municipal Law Review 33:342). In essence, the uniform citation is similar to a speeding ticket issued to the violator of an ordinance (or state statute). Depending on the enabling language, the violator usually signs the citation, posts a bond in accordance with an established schedule and agrees to appear in court to enter his plea or forfeit his bond.

If the violator choses to forfeit the bond which he has posted, generally no court appearance is made (though the court may refuse to accept the bond and issue a summons or warrant for the violator). However, if the violator wishes to defend against the charge, he may appear in court, enter his plea and be assigned a court date for his trial.

The principal advantage of the uniform citation and complaint is that the local or state regulatory official now has a means to begin an enforcement action (law suit) against the violator immediately. The action is commenced simply with the issuance of the "ticket" without going through the regulatory authority's attorney (district attorney, county attorney, attorney general, etc.). The obvious benefit is the avoidance of delays typically experienced when seeking action by the authority's attorney.

It must be noted that the uniform citation and complaint is used only to commence the enforcment action. If the violator wishes to plead not guilty, time for the trial must be scheduled on generally crowded court calendars. The regulatory authority's attorney also must find time to prepare and present the case at the trial. Thus, enforcement delays are not avoided completely if the violator pleads not guilty.

Delays are avoided if the violatory pleads guilty, but in such instances, the authority has succeeded only in collecting a forfeiture. No injunction or other equitable relief may be imposed. Therefore, if the regulatory authority desires injunctive relief, use of the citation and complaint poses a serious disadvantage. The violator could chose to post and forfeit a bond and continue the violation.

Wisconsin has recently enacted statutory language which enables local regulatory officials to use the citation and complaint process against violators of on-site system ordinances and rules. Section 66.119 of the Wisconsin Statutes (1975) provides the following:

> 66.119 Citations for certain ordinance violations
>
> (1) Adoption; content. (a) the governing body of any county, city or village may by ordinance adopt and authorize the use of a citation to be issued for violations of ordinances other than those for which a statutory counterpart exists.
>
> (b) An ordinance adopted under par. (a) shall prescribe the form of the citation which shall provide for the following:
>
> 1. The name and address of the alleged violator.
> 2. The factual allegations describing the alleged violation.
> 3. The time and place of the offense.
> 4. The section of the ordinance violated.
> 5. A designation of the offense in such manner as can be readily understood by a person making a reasonable effort to do so.
> 6. The time at which the alleged violator may appear in court.
> 7. A statement which in essence informs the alleged violator:
>
> a. That * * * the alleged violator may make a cash deposit of a specified amount to be mailed to a specified official within a specified time.
>
> b. That if * * * the alleged violator makes such a deposit, * * * he or she need not appear in court unless * * * susequently summoned.
>
> c. That * * * the alleged violator makes a cash deposit and does not appear in court, * * * either he or she will be deemed to have tendered a plea of no contest and submitted to a forfeiture not to exceed the amount of the deposit or * * * will be summoned into court to answer the complaint if the court does not accept the plea of no contest.
>
> d. That if * * * the alleged violator does not make a cash deposit and does not appear in court at the time specified, an action may be commenced against * * * the alleged violator to collect the forfeiture.
>
> 8. A direction that if the alleged violator elects to make a cash deposit, * * * the alleged violator shall sign an appropriate statement which accompanies the citation to indicate that * * * he or she read the statement required under subd. 7 and shall send the signed statement with the cash deposit.
> 9. Such other information as may be deemed necessary.
>
> (c) An ordinance adopted under par. (a) shall contain a schedule of cash deposits which are to be required for the various ordinance violations for which a citation may be issued. The ordinance shall also specify the court, clerk of court or other official to whom cash deposits are to be made and shall require that receipts be given for cash deposits.
>
> (2) Issuance; filing. (a) Citations authorized under this section may be issued by law enforcement officers of the county, city or village. In addition, the governing body of a county, city or village may designate by ordinance or resolution other county, city or village officials who may issue citations with respect to ordinances which are

directly related to the official responsibilities of the officials. Officials granted the authority to issue citations may delegate, with the approval of the governing body, the authority to employees. Authority delegated to an official or employe shall be revoked in the same manner by which it is conferred.

(b) The issuance of a citation by a person authorized to do so under par. (a) shall be deemed adequate process to give the appropriate court jurisdiction over the subject matter of the offense for the purpose of receiving cash deposits, if directed to do so, and for the purposes of sub. (3)(b) and (c). Issuance and filing of a citation does not constitute commencement of an action. Issuance of a citation does not violate s. 946.68.

(3) Violator's options; procedure on default. (a) The person named as the alleged violator in a citation may appear in court at the time specified in the citation or may mail or deliver personally a cash deposit in the amount, within the time and to the court, clerk of court or other official specified in the citation. If a person makes a cash deposit, * * * the person may nevertheless appear in court at the time specified in the citation, provided that the cash deposit may be retained for application against any forfeiture which may be imposed.

(b) If a person appears in court in response to a citation, the citation may be used as the initial pleading unless the court directs that a formal complaint be made, and such appearance confers personal jurisdiction over the person. The person may plead guilty, no contest or not guilty. If the person pleads guilty or no contest, the court shall accept the plea, enter a judgment of guilty and impose a forfeiture. A plea of not guilty shall put all matters in such case at issue, and the matter shall be set for trial.

(c) If the alleged violator makes a cash deposit and fails to appear in court, the citation may serve as the initial pleading and the violator shall be deemed to have tendered a plea of no contest and submitted to a forfeiture not exceeding the amount of the deposit. The court may either accept the plea of no contest and enter judgment accordingly or reject the plea. If the court accepts the plea of no contest, the defendant may move within 10 days after the date set for * * * the appearance to withdraw the plea of no contest, open the judgment and enter a plea of not guilty if * * * the defendent shows to the satisfaction of the court that * * * the failure to appear was due to mistake, inadvertence, surprise or excusable neglect. If the plea of no contest is accepted and not subsequently changed to a plea of not guilty, no costs or fees shall be taxed against the violator. If the court rejects the plea of no contest or if the alleged violator does not make a cash deposit and fails to appear in court at the time specified in the citation, an action for collection of the forfeiture may be commenced. A city or village may commence action under s. 66.12(1) and a county may commence action under s. 288.10. The citation may be used as the complaint in the action for the collection of the forfeiture.

(4) Relationship to other laws. The adoption and authorization for use of a citation under this section shall not preclude the governing body from adopting any other ordinance or providing for the

> enforcement of any other law or ordinance relating to the same or any other matter. The issuance of a citation under this section shall not preclude the proceeding under any other ordinance or law relating to the same or any other matter. The proceeding under any other ordinance or law relating to the same or any other matter shall not preclude the issuance of a citation under this section.

It is likely that the use of the uniform citation and complaint will result in an administrative cost savings since the amount of time spent by the attorney involved should be less. Revenue should be generated by forfeited money, as well.

Small claims courts--Most states have small claims courts for cases involving less than 500 or 1000 dollars. Usually, these courts allow an abbreviated, less formal procedure. Standardized forms are often used. When seeking only fines or forfeitures, it is recommended that state and local authorities consider using small claims courts to prosecute on-site system ordinance and rule violations. Regardless of how the enforcement action was commenced (e.g., uniform citation and complaint, summons, warrant), if the regulatory agency is seeking a forfeiture less than the dollar limit set for the court, usually it may be brought in the small claims court.

In addition to the simplified procedures and standardized forms, small claims courtsalso have the advantage of typically having less of a case backlog. Thus, enforcement of violations can be accelerated at less cost. The only disadvantage is the unavailability of injuncture or equitable relief in the courts.

Civil service status--Many local regulatory officials and some state officials serve at the pleasure of those who appointed them to their jobs. Stewart (1974) noted that Wisconsin regulatory officials are sensitive to this and have expressed a need for increased job security. These statements are probably typical of those from officials of other states. In some cases, the lack of job security probably has hindered vigorous application and enforcement of on-site system regulations. To provide the necessary job security, it is recommended that a civil service classification be established for local and state officials responsible for the implementation and enforcement of sanitary codes.

Operation and Maintenance--

This phase of on-site system regulation is often the most overlooked. For the conventional septic tank system, it is recommended that the system be inspected annually and the septic tank be pumped when necessary (USPHS, 1967). Many states make this recommendation in their codes. However, this recommendation is seldom followed since there is typically no regulatory enforcement of these provisions. For example, Wisconsin has requirements in the state code which require the pumping of septic tanks whenever the combined depth of sludge and scum equals one-third of the effective depth of the tank (Wis. Admin Code Chap. 62.20). After surveying septic tank systems serving homes around 8 lakes in Wisconsin, the state Division of Health concluded there was "an almost complete lack of servicing of the septic tanks." This survey concluded that this lack of maintenance likely resulted because few property owners knew the location of their systems (Wirth and Hill, 1967).

From the homeowner's perspective, a regulatory program will be necessary, because all too often, the system is approached with an out-of-sight-out-of-mind attitude. Winneberger (1976) has isolated two reasons for this neglect:

1. The tolerance of many septic tank systems to function faithfully without any maintenance, and

2. The system resists maintenance in that it is buried underground.

For systems other than conventional septic tank systems, maintenance requirements may be more involved. It seems certain that a regulatory program involving more than a recommendation for septic tank cleaning in a local ordinance or state code will be needed to assure adequate maintenance.

There are two components necessary for a successful regulation of the maintenance of any type of on-site system. One component must assure that the location of the system is known. The location must be referenced to a benchmark or other permanent fixture or marker to allow ease of location after the system is underground. Also, a filing and retrieval system must be established to provide information about the system's location whenever future maintenance is to be performed.

The other component of a successful regulatory program should provide a method of assuring that each system will be inspected and maintenance performed when needed. This may be accomplished in one of several ways. The most preferred method is the maintenance permit program, in the author's opinion.

Maintenance permit--It is recommended that regulatory agencies establish maintenance programs to assure that systems are inspected regularly and serviced when necessary. To guarantee this, the program would require periodic inspection of the system as a prerequisite to issuing (or renewing) a maintenance permit. The system owner would be mailed a maintenance permit application reminding him to have his system inspected and have any necessary servicing performed within a specified time period (e.g., 60 days). The person making the inspection would sign and date one portion of the owner's permit, thereby certifying that inspection and servicing was completed. Just prior to the expiration of the permit period the process would be repeated.

Under this program it would be necessary that system inspections be performed by individuals knowledgeable with the operation of such systems. Wastewater pumpers or haulers could perform inspections of septic tank systems in states where they are licensed. Systems with mechanical components could be inspected by licensed plumbers, installers or public inspectors. Alternatively, special purpose districts could perform inspections and maintenance as a service to system owners in the district. For example, the Santa Cruz Septic Tank Maintenance District in California currently provides this service.

The administration of this permit program could be a simple routine matter involving clerical staff time only. The clerk would mail out the permit application forms and issue the permit upon receipt of a completed application and fee. The clerical staff would maintain the files which would have a "tickler" system to retrieve those permits nearing expiration.

This program could be applied most easily to systems installed since the date of the enabling ordinance or statute. Lack of knowledge about the systems and the owner's address would usually make it difficult to impose the permit program upon existing systems. Newly-installed systems would probably not need maintenance and could be issued the first permit simply upon payment of the fee.

The enabling ordinance or statutory language which establishes this permit program must provide that it is unlawful to occupy a home served by an on-site system unless a current maintenance permit has been issued for that system. Thus, when the owner failed to renew a maintenance permit, he would be in violation of the ordinance or statute. From a legal viewpoint, enforcement of this type of violation is straightforward since the only fact which has to be proven is that the owner (or others) occupies a home served by an on-site system which does not have a valid maintenance permit. It is not necessary to prove the negative facts that the system was not inspected, was not maintained or was not adequately functioning. The ease of the proof of facts should encourage enforcement and prosecution by governmental legal officials (district attorneys, etc.) and, depending upon the wording, the courts might give equitable relief by ordering inspection of the system at the owner's expense, as well as ordering the payment of a fine or forfeiture.

Stewart (1976) proposed the following model statutory language suitable for any local or state regulatory authority which desires to establish a septic tank maintenance permit program:

> MODEL SECTION 1.0 SEPTIC TANK MAINTENANCE PERMIT
> 1.1 PERMIT REQUIRED. No owner may occupy, permit to be occupied, rent, lease, live in or reside in, either seasonally or permanently, any building, residence, or other structure serviced by an on-site domestic sewage treatment and disposal system; unless the owner has a valid septic tank maintenance permit for that system issued in his name by the _______________ (sanitary inspector or zoning administrator). Owner is defined to mean a natural person, partnership, corporation, the state or any subdivision thereof.
>
> 1.2 FEE. A fee of $ ______ shall accompany each application for the septic tank maintenance permit.
>
> 1.3 PERMIT APPLICATION. Application for a septic tank maintenance permit shall be made to the ____________ (sanitary inspector or zoning administrator) on forms supplied by him. All applications shall state the owner's name and address, the address or location of the private sewage system, and shall contain the following statement:

"I certify that on ___ day of _______, 19__, I inspected the septic tank located at the address stated on this application, and I (check one):

_______ pumped all sludge and scum out of the septic tank, or

_______ found that the volume of sludge and scum was less than 1/3 of the tank volume, and I did not pump the septic tank.

____
Signature

Certification or license number _______________________

1.4 ISSUANCE. The __________ (sanitary inspector or zoning administrator) shall issue a permit to the applicant upon receipt of the fee and a completed application, properly signed by a person licensed to service septic tanks and stating his sanitary license number. The permit shall include on its face all information contained in the application and shall contain the date of issuance.

1.5 VALIDITY. The permit issued under this section shall be valid for a period of two years from the date of issuance.

1.6 SALE OF PROPERTY. When property containing a private domestic sewage system is sold, the new property owner, prior to occupying, renting, leasing, or residing in the building, residence or structure served by the system, shall make application for and receive a septic tank maintenance permit; however, the system may be used for a period not to exceed 30 days during pendency of his application for the permit.

Additional model language is suggested to require that information about the location of all systems installed in the future. It is recognized that many jurisdictions already have this requirement and the following model language is suggested for those which lack such a requirement:

MODEL SECTION 2.0 PLAN VIEWS

2.1 Every application for a sanitary permit shall include a detailed plan view of the proposed system prepared or drawn by a __(state)__ Registered Surveyor or a __(state)__ Professional Engineer. The plan view shall be signed by said surveyor or engineer and shall also contain the license number of said surveyor or engineer.

2.2 This detailed plan view shall be dimensioned and drawn to scale and shall show the location of the system and the dwelling served by such system. The recommended scale is _____ _____ _____ but in any case the scale used shall be sufficient to show clearly all the required dimensions and distances enumerated below.

2.3 The following dimensions and distances shall be shown on the plan view: the dimensions of the entire lot or a sufficient portion such that all other required dimensions and distances may be shown; the dimensions of the dwelling to be served by the system; the location of the dwellings and all other buildings on the lot with distances from lot lines to said dwellings and buildings; the location of all septic tank and other treatment tank manholes and the distance and direction of each manhole to the dwelling and to any other nearby reference points; the location and dimensions of all soil absorption fields and replacement areas; and the location and distances from all wells, reservoirs, swimming pools or high water marks of any lake, stream, pond or flowage located on the lot or on adjacent properties within 100 feet of the septic tanks, treatment tanks, sewage disposal systems or replacement disposal areas.

2.4 No on-site sewage treatment and disposal system shall be installed, modified, added to or replaced unless a plan view for that system drawn by a registered surveyor or professional engineer has been submitted.

To be useful, this information about location must be retrievable and the following model language is offered to establish a filing system:

MODEL ORDINANCE SECTION 3.0 PLAN VIEW FILING SYSTEM 3.1 A filing system for plan views of on-site sewage treatment and disposal systems is hereby established.

3.2 It shall be the duty of the ________ (county zoning administrator, sanitarian or other designated person) to accept all approved plan views and to file them by the address or the location of the system. He shall further establish a cross-index which shall list the original owner's name and shall cross list the address of the system. Further, he may establish any additional files or other cross-indexes which he determines advisable. In furtherance of this filing system the ________ (county zoning administrator, sanitarian or other designated person) may require that additional information shall be included on the plan view to aid in filing, indexing or retrieving said plan view.

Conditional sanitary permit--As an alternative to the maintenance permit program, sanitary permits for on-site systems could be made valid subject to the condition that inspection and pumping (if necessary) be performed on a specified regular basis. The enabling legislation or ordinances would have to be worded to make it unlawful for a system owner to use his system unless he had a valid sanitary permit.

Local filing requirement--The ability to locate the components of an on-site system is an obvious prerequisite to any inspection and maintenance program. Some state and local authorities currently require the filing of a proposed (or as-built) plan of the system. Summit County Health Department in Ohio has such a filing requirement and maintains an excellent retrieval system. If a septage pumper is having difficulty locating a septic tank, the Department is able to give detailed location information over the phone. Similarly, the State of Wisconsin requires that every county in the state must file a plan for each system installed in its jurisdiction and "cross-index" the plan to facilitate retrieval. Dane County, Wisconsin also requires a copy of the plan with an adhesive back to be applied to the building's electric fuse or circuit breaker box.

Thus, it is recommended that every regulatory authority require that each system owner file an "as-built" plan of his system, clearly referencing the location of the system components. Such a plan is invaluable when it becomes necessary to inspect or service the on-site system. Many owners do not know the location of their systems and this makes maintenance difficult. In order to improve this phase of regulation, it is recommended that states and/or local authorities adopt this filing requirement and establish a file for these plans indexed by street address, name of original owner, installer and, perhaps, legal description.

Failing Systems--

Sanitary surveys--Detection of the failing system is one of the most important aspects of this final regulatory phase. State and local authorities should seek funding to perform sanitary surveys. Although a large staff and budget may be necessary, these surveys are the most thorough method of determining which systems are failing.

Sanitary surveys may be conducted at one of several different levels of sophistication as follows:

1. A visit to the location of each system to check for physical signs of failure (i.e., odor or wetness).
2. The owners or users of the system might be questioned about the operation of their system and a search of the regulatory agency's records made to obtain information about the individual system (age of the system, maintenance, location, etc.).
3. Each system is checked for failure by flushing an indicator dye through the system (dye testing).
4. Attempts made to check on the adequacy of the site selection and installation of the system (by analyzing the soil and digging up a portion of the system) and on the effect of the system on the groundwater (by taking well samples).

Sanitary surveys provide information which should permit an estimation of the regulatory program effectiveness. Obviously, the greater the level of sophistication used in performing the survey, the better this estimation.

The commitment of regulatory agency staff time to perform a sanitary survey, even at lower levels of sophistication, is generally so great that it tends to limit the number of sanitary surveys performed. In a 1967 survey of the individual on-site sewerage systems around eight lakes, the Wisconsin Department of Health and Social Services (DHSS) estimated that it expended over four staff hours per system investigated to inspect, compile and write-up the survey of 410 systems (Wirth and Hill, 1967). Thus, a major limitation on the use of sanitary surveys is the large expenditure of time required. Although, it is possible that the agency could obtain part-time help to perform these surveys (i.e., students or National Guard or Armed Forces Reservists).

An equally important limitation of sanitary surveys is the difficulty of determining the actual cause of failure, since the system is buried. The cause of failure is important because it tends to point out weaknesses in the regulatory program itself. This type of information sometimes may only be obtained by excavation of a portion of the failed system. The expense of this type of survey is great.

Also, the use of surveys are limited because of difficulty in determining when failure occurs. One type of failure (hydraulic) can be easily detected while others (treatment efficiency) require a detailed survey of the groundwater surrounding the failed system. In hydraulic failure, the sewage does not infiltrate into the soil, but instead earlier backs up into the home or ground surface, collects in depressions, runs off, or evaporates. This may pose public health problems and the possibility for degradation of surface water quality.

In treatment efficiency failure, the system fails to adequately treat the sewage before it reaches the groundwater. While not as obvious, this inadequate treatment must be considered as a failure. Where individual water supply systems are used, the threat to public health is apparent if pathogenic material is permitted to reach the groundwater. Also, the groundwater is degraded by the addition of pathogenic and nutrients contained in domestic sewage.

Other limitations arise due to fluctuations of rainfall and groundwater levels. For example, a system may be adequately functioning during dry seasons but fail when the rainfall and groundwater levels are high in the spring. Thus, the time of the year that a survey is performed may affect the results of the survey.

Violation as an encumbrance--Often, the effect of a sanitary code violation on the property title is unclear. In an effort to give notice of sanitary violations to potential buyers of land, it is recommended that state or local governmental units adopt, either legislatively or administratively, the requirement that regulatory officials must file copies of all violations with the register of deeds or similar official. Reported violations would have to contain the legal description of the property on which the violation is occurring.

The effect of such a filing requirement would be to have the violation appear in the chain of title whenever an abstract or title insurance policy is prepared. If the owner of the violating system ever attempted to obtain an additional mortgage on his property or to sell his property, the potential mortgagee or buyer would be alerted to the violation. With the violation appearing as a possible cloud upon his title, the property values likely would be depressed, providing the landowner with an incentive to correct the violation.

As an alternative or adjunct to this filing requirement, notice of existing violations could be given to potential buyers by requiring an inspection of the system prior to sale. However, due to constitutional limitations, it may not be possible to legislatively bar the sale of property until any violations are corrected. Despite this limitation, the pre-sale inspection would accomplish its primary purpose of giving notice to buyers.

Centralized Management of On-Site Systems

The use of centralized management of on-site systems probably offers the best regulatory technique available. Centralized management is the exercise of administrative and/or regulatory control by a single authority over dispersed on-site systems. While this concept is quite simple, it does require the existence of an entity which has the authority to perform certain functions.

Powers Needed by a Management Entity--

Any management entity which endeavors to administer on-site systems with the same effectiveness as one which manages the conventional central system, must have the power and authority to perform vital functions. First, the public management entity should be empowered to own, purchase, lease and rent both real and personal property; and to plan, design, construct, inspect, operate and maintain all types of on-site systems located within the jurisdiction of the management entity, whether the system is a typical septic system serving a single family residence or a much more involved, complex system serving a group of residences. This does not imply that the entity should be limited to providing services within its jurisdictional boundaries, only that the entity should clearly have the above "ownership and operation" powers within its boundaries. The entity may be given by state statute, by case law, or as terms under a contract, extra-jurisdictional authority to operate, maintain and perhaps own such systems outside of the entities boundaries as well.

Secondly, it is highly desirable that the entity meet the eligibility requirements for loans and grants in aid of construction of these systems from both the federal and state governments. While it is obvious that a management entity can function without being eligible for these loans and grants, the viability of the centralized management is strengthened when grant money is used to offset some or most of the costs to the families served by the entity. This is especially true since low income rural families often cannot afford to finance the entire cost of their system. Experience has shown that low-income families cannot pay wastewater bills in excess of $7.00 per month or a combined water-sewage bill of $14.00 per month (Commission on Rural Water, 1973). This rate is difficult to reach without benefit of public subsidy. Non-rural residents have typically paid considerably less than this amount.

Third, the mangement entity should be able to enter into contracts, to undertake debt obligations either by borrowing and/or by issuing stock shares or bonds, and to sue and be sued. These powers are more than mere legal niceties because without them the entity would not be able to acquire the property, equipment and supplies and services necessary to construct or operate the on-site systems.

Fourthly, the entity must be able to fix and collect charges for sewerage usage, determine the benefit to all property in its jurisdiction, set the value or cost of such benefit and assess or collect the cost from each property owner so benefitted. Further, it has been argued that the entity should have the power to levy taxes upon all owners within its jurisdiction for the purpose of raising funds to administer the program. Obviously, this taxing power is limited to various governmental or quasi-governmental management entities. In lieu of taxing power, the non-government management entities must have the authority implied or directly granted, to set and charge user fees to cover administrative costs.

Fifth, and quite important, the entity must have the power to plan and control how and at what time service will be extended to those within its jurisdiction.

Lastly, the entity would be much more effective in protecting the public health and promoting good public sanitation if it were also empowered to make rules and regulations regarding proper sanitation and the use of on-site systems and to issue orders against violators of these rules or regulations. As a desirable additional power, the entity should be empowered to require the abatement of malfunctioning systems and to require the replacement of all such systems, according to the plans of the entity. Again, several of these powers would only be available to governmental or quasi-governmental management entities.

Various Types of Public Central Management--
The entities which could provide central management of on-site systems vary from state to state. State constitutions, state statutes, administrative agency rules and regulations must be examined to determine which types of entities are authorized to manage on-site systems on a state by state basis. Further, the case law (essentially laws made by the courts) must be checked to determine if the courts have construed the constitution, statutes or regulations to give or remove the authority to manage such a system from a candidate entity.

The following discussion of various entities does not attempt to identify which entities are permitted in each state. A sample of some of the possible entities is as follows:

Municipalities--While this term has many and various legal definitions, it is used here to include only incorporated cities and villages. In some states, the municipal charter granted to the city or village authorizes the administration of water and sewer services as a permissible governmental activity. Other states provide this same authorization to cities and villages in the state statutes dealing with municipal law. Generally, these statutory provisions detail the procedures to be followed in supplying these services. Thus, both the municipal charter and any and all applicable municipal law statutory language must be checked to determine the extent of this entity's authority to own and operate on-site systems. This entity typically has the authority to provide centralized management.

Counties and townships--While the general authority of counties in the U.S. ranges from a boundary drawn on a map (New England States) to complete home rule powers bordering on that of a sovereign, the county may be empowered to own and operate on-site systems. The authority of both counties and townships is set out in each state's statutes and laws. It is necessary to check the statutory language to determine whether either has been granted sufficient authority.

Special purpose districts--There are many special purpose districts which are given the requisite authority to properly perform as central management entities. These districts are quasi-governmental in nature and their authority is usually set out in the state's statutes or laws. Because the district is often included in the statutory definition of "municipality", however, the authority of such a district is expanded. Hence, it is necessary to examine the state's statutes on municipal law to determine the real extent of the district's power.

Single purpose special districts, established to deal with public sanitation, will generally have sufficient authority. Also, multi-purpose districts having a primary purpose other than public sanitation might also have sufficient authority to satisfactorily own and operate on-site systems. While districts may have many different names, e.g., sanitation district, service district, sanitary district, etc., the authority of the district is determined by the underlying statutory language.

Private non-profit corporations--These entities must be incorporated as non-profit corporations in the state in which they seek to perform management functions. Depending on the laws of the individual state and the services to be provided, these corporations may be considered to be public utilities and as such, would have to comply with the laws and regulations of the state's public service or public utility commission. The authority of this type of entity would be contained in its charter of incorporation and in the applicable public utility law of that particular state.

Rural electric cooperatives--In some states the REA cooperatives are authorized to perform the functions necessary for proper administration of on-site systems. The authority of these cooperatives is contained, in part, within the state's statutes.

Private profit-making businesses--This type of entity may be either a sole proprietorship or incorporated business formed to supply sanitation services. Regardless of the type of business, the state public service or public utility commission usually regulates this type of entity. The authority of the private profit business is limited by any public utility laws or regulations of the commission which apply to this type of entity.

Others--There are a few other types of entities which might be authorized in some states to own and operate on-site systems. For example, systems installed on Native American reservations would probably not fall within any of the first six types of entities but, instead, would be controlled by the U.S. Public Health Service. Further, some states, such as Wisconsin, have a strong history of cooperatives and may permit a co-op (other than a REA) to function as a mangement entity.

Examples of Central Management of On-Site Systems--

While new in concept, central management of on-site systems has been tried in several places. The following discussion of examples is not exhaustive, but is intended to be illustrative of the state of the art in central management. In at least half of the following examples, the management entity is public or quasi-public; however, few of these entities have gone the full distance of both owning and operating the on-site systems.

Most of the case histories are drawn from California, Wisconsin and those project areas affiliated with the National Demonstration Water Project (NDWP). The reason that California is in the vanguard of the movement to centrally managed on-site systems in rural areas is due, in part, to the efforts of Winneberger, septic tank consultant. Winneberger has been an advocate of the septic tank system as an alternative to central collection and treatment systems. District or public management of the septic tank systems is essential to this concept (Winneberger and Andermann, 1972).

The NDWP recognizes centrally managed on-site systems as the preferred alternative to central collection and treatment systems in rural or sparsely populated areas. The NDWP developmental activities were originally funded by the Office of Economic Opportunities (OEO) and was founded to establish a method of rural water development. The developed method consisted of central management of on-site systems (individual wells and wells serving clusters of residences) of water supply. Since the original development of rural water systems, the NDWP has expanded its concern to the development of a method of rural wastewater treatment and disposal. Essentially, the method consists of the same central management of on-site systems. However, the NDWP has shown some hesitancy to rely on public central management entities; instead, it espouses the use of private entities, especially non-profit corporations.

To date, the experience in Wisconsin has been somewhat limited. Public central management in the form of special purpose districts, has been used to inspect and maintain septic tanks within the district's jurisdiction. Currently, however, there is only one district which both owns and operates on-site systems (Otis, 1977).

A possible fourth category of experience in central management of on-site systems has arisen due to the demonstration of low pressure or vacuum sewers. While it can be argued that pressure sewers are really central systems, there is an additional component which tends to put pressure/vacuum sewers under the rubric of on-site systems. In the case of pressure sewers, this component is the effluent or grinder pump. These components are installed in individual residences or in some cases, a single pump may be used to serve a cluster of homes. However, ownership, maintenance and operation of these pumps is handled by a central management entity and the similarity to on-site treatment and disposal systems is obvious.

The examples of central management of non-central systems are as follows:

California--

Santa Cruz County septic tank maintenance district--As the name implies, the primary function of this entity is the inspection and pumping of all septic tanks within the district. The county board of supervisors is required to contract out the inspection and pumping services. The district is empowered to establish a monthly charge and to collect this charge by separate billing or through taxes. Provision is made such that the system owner will bear the cost of "exceptional" pumping (defined as more than one pumping in any consecutive 3 year period) as well as the cost of repairing or replacing the system.

This district is not given the authority to own systems and, as of 1974, it did not perform soil studies of individual sites nor did it design systems. Without actual ownership of the systems, the district is ineligible for most construction grants or loans and by not providing individual site evaluation and system design, the district loses control over the effectiveness and reliability of the systems it seeks to maintain. This burden falls upon the county health department. Thus, this district is somewhat limited.

Georgetown Divide Public Utility District (GDPUD)--This district is located in El Dorado County, California, and employs one full time environmentalist. By legal arrangements (formation of a special sewer improvement district within the GDPUD) the Auburn Lake Trails subdivision is receiving central management services from the GDPUD. The district's environmentalist is authorized to:

1. Perform feasibility studies on lots within the subdivision to evaluate the potential for the use of individual on-site systems;

2. Design specific kinds of on-site systems to serve the individual sites;

3. Monitor the installation of all systems within the subdivision;

4. Inspect and maintain the systems after installation; and

5. Monitor water quality to determine the effect of the individual systems upon water leaving the subdivision.

This district also may require the installation of public sewers when and where necessary. Currently, one area of the subdivision is already sewered. More sewers may be required, but the build out rate of this recreational subdivision is about 3 percent per year, thus the need for additional sewers will be determined sometime in the future. The functions performed by this district border on almost complete central management of on-site systems. The only limitation appears to be the fact that the district does not own the individual systems.

Bolinas Community Public Utility District (BCPUD)--This district was involved in several legal problems in 1974. At that time, the district was in the process of constructing a central system to treat the wastewater from the populous area of the district (Bolinas CBD). Individual on-site systems were strongly favored by the BCPUD for use in the less densely populated areas. The district had performed several management functions such as surveying all existing systems within the district and establishing design requirements. It proposed to monitor the construction of all systems, maintain the systems and monitor water quality in the district's watershed. Further, in an effort to preserve and protect the effectiveness of the area for individual soil disposal systems, the district proposed to require permits for excavation, filling, or grading within the district. However, there has been a legal question involving the effect of overlapping county (Marin) and district jurisdiction in the area of individual on-site systems. Lack of resolution of this question as well as the additional requirement of statisfying the State Water Quality Control Board has impeded, to some degree, this district in the management of individual on-site systems. However, it is anticipated that this district will approach the GDPUD in the degree of central management of individual systems, lacking only ownership of the systems themselves.

National demonstration water projects--

Guyandotte Water and Sewer Development Association (GWSDA)--This program, located in Logan County, West Virginia, will involve several discrete individual projects. The first project planned is to supply both water and sewer service to 250 families in Big Creek. The proposed wastewater treatment system does not contemplate the use of individual systems, but the use of a combination of gravity and pressure lines and a single central treatment facility. The entity used to manage the systems is a public service district as provided for by West Virginia law. Other public service districts have been approved for additional projects in the Guyandotte area with the GWSDA Association contracting to provide operation and maintenance services to each of the public service districts. It is this central management provision for operation and maintenance that makes this program of such interest. In effect, what is proposed is provision of central management services to a group of discrete, separate central systems.

Lee County Cooperative Clinic--This program is administered by a rural health facility. There are 4 development areas in the current plan. The Poplar Grove area is a community which is experiencing a severe health problem due to septic tank system failures. A sewer improvement district has been formed to administer the Poplar Grove project. The improvement district hopes to qualify for an EPA construction grant to construct a conventional system. Again, despite the fact that the program proposes to use a central system, it is of interest since the Lee County Co-op apparently will provide central management services to several discrete, separate (albeit conventional) central systems.

Cooperatives Water and Sewer Association (CWSA)--This association consists of a partnership of 4 rural electric cooperatives which serve 18 counties in the northwest Florida panhandle. A recent amendment to the Florida statutes permits these co-ops to own and operate both water and sanitary sewer systems. There are several advantages of extending the vast experience of the co-ops in the field of rural electrification to managing water and sewer systems. For example, they already have the managerial expertise to bill for services and supply operation and maintenance services. Also, in most cases, the addition of other utilities (water and sewer) to their basic responsibility of supplying electricity would be the most cost effective method of supplying these additional services to the rural areas.

As noted previously, rural electric cooperatives might not have the authority to perform the necessary management functions. The state laws must be examined. This CWSA program certainly warrants further attention due to the natural meld of sewer and water supply with the existing co-ops.

Wisconsin--

Town Sanitary Districts (TSD)--Wisconsin has given sufficient authority to various public districts and has empowered them to own, operate and maintain various systems of sewage treatment and disposal including on-site systems serving individual residences. One such type of district is the Town Sanitary District (TSD). In a thorough survey of Wisconsin's Town Sanitary Districts, performed in 1974, no districts reported that they owned any individual systems; however, the survey did disclose that some do perform management functions such as monitoring the installation of new septic systems (13% of those responding to the questionnaire); inspecting the systems (16%) and maintaining-pumping the systems (4%) (Klessig and Yanggen, 1974). Since the survey, one town sanitary district Sanitary District No. 1 of the Town of Westboro, Taylor County, Wisconsin, has acquired ownership of septic tanks located on private property. The tanks serve as pretreatment to the wastewater collection in small diameter gravity sewers. This system has been in service since June 1977 and is functioning well.

REGULATION OF ALTERNATIVE SYSTEMS

The scope of this discussion is limited to on-site treatment and/or disposal systems which handle only small wastewater flows. Unfortunately, there is no common terminology used in the field of on-site treatment and disposal, hence, definitions are necessary. The following are offered to aid in this discussion.

Definitions

Conventional Treatment--

"Conventional treatment" is that which is obtained by using the septic tank as it is generally used to treat wastewater. Although there is a wide variance in state septic tank design parameters, this variance is not of concern here as it is not necessary to examine the specific design parameters in consideration of regulatory programs.

Alternative Treatment--

"Alternative treatment" is defined as any treatment method other than "conventional" septic tank treatment. No value judgment is intended by the choice of these two words, i.e., alternative treatment methods are not inherently better (or worse) than the conventional treatment method.

Conventional Disposal--

"Conventional disposal" means the method of disposal which relies on subsurface soil infiltration of the treated wastewater. Typical disposal methods are soil absorption trenches, beds, seepage pits, etc. Henceforth, these will be simply referred to as subsurface soil absorption fields. Again, it is noted that there is a wide variance in the design and site evaluation criteria for these fields but this variance has no direct impact in this discussion.

Alternative Disposal--

The easiest wasy to define "alternative disposal" methods as used here is simply to say that it includes all disposal methods other than conventional disposal. As with alternative treatment, no judgment value is intended by this bifurcation of disposal methods into conventional (subsurface soil absorption) and alternative (all other methods of disposal).

Surface Discharge--

"Surface discharge" is defined as the discharge of effluent from a wastewater treatment system into a receiving body of water. Although PL 92-500 does not specifically define "surface discharge", section 502 (12) of the act defines discharge of pollutants to have a meaning quite similar to "surface discharge" as defined here, since "pollutant" is defined by section 502 (6) of the act to mean sewage, sewage sludge, chemical wastes, biological materials, etc. discharged into the water.

Point Source--

"Point source" as defined by section 502 (14) of PL 92-500 is any discernible, confined and discrete conveyance from which pollutants are or may be discharged.

Publically Owned Treatment Works--

"Publically owned treatment works" are any device or system used in the storage, treatment of municipal sewage which is owned and operated by a public entity. This is the same meaning as provided in Section 212 (2) of PL 92-500. If publically owned, a system serving more than two residences would be included within this classification.

Central Management of Non-Central Systems--

A number of on-site treatment and disposal systems serving individual residences and/or small clusters of homes and non-residential structures is referred to as a "non-central system." "Central management" of this non-central system is defined to mean a single governmental or quasi-governmental unit having the authority to manage these dispersed systems. Thus, "central management of non-central systems" is simply a regulatory management technique as discussed previously. The use of central management will be suggested as a regulatory technique for most of the alternative treatment-disposal systems analyzed in this Appendix.

Matrix of Permissible Treatment-Disposal Combinations

A matrix of possible treatment and disposal combinations of on-site sewerage is presented in Table D-8. Any system may be thought of as having a treatment and a disposal aspect. Essentially, any system, whether available today or yet to be developed and/or proven , will have either a conventional innovative or no treatment method coupled with conventional, innovative or no disposal (containment). However, a further breakdown of alternative disposal methods is necessary to adequately set out all of the possible alternative disposal methods. The three additional subheadings: (1) soil, (2) surface water discharge and (3) evapotranspiration (E-T).

An attempt was made to include, in the appropriate locations in this matrix, descriptions of a few of the known on-site treatment and/or disposal combinations. It is believed, however, that any other combinations can be added. The only function served by this matrix is to aid in the creation and discussion of regulatory programs to control various types of on-site systems which have been or could be developed.

Elimination of Certain Treatment - Disposal Combinations--

For the purpose of this discussion, certain combinations of treatment-disposal combinations have been dismissed as being either highly improbable and/or undesirable. The conventional treatment - containment and off-site disposal is considered to be unlikely because it is doubtful that any purpose would be served by conventional (septic tank) treatment of wastewater prior to discharge into a holding tank. The only possible justification for treatment of the wastewater would be to increase the likelihood that municipal wastewater treatment plants would accept the pumpage from the holding tanks. All of the combinations providing no treatment prior to disposal were dismissed because they are either highly undesirable or unlawful.

Regulation of Alternative Soil Disposal Systems

Since many alternative on-site systems are more complex than the septic tank-soil absorption system (e.g., mounds, evapotranspiration systems, etc.) there are increased possibilities for error in their design and installation. Thus, the regulatory program must be capable of adequate plan review and installation inspection.

Suggested Regulatory Techniques--

Pre-approval functions--The regulatory process should be initiated either by the submission of plans for review and approval or by application for a permit to install an alternate system. Generally, every on-site regulatory program would impose either the requirement of obtaining plan approval or permit prior to commencing construction of any of these systems.

TABLE D-8. MATRIX OF ANTICIPATED POSSIBLE TREATMENT AND DISPOSAL COMBINATIONS OF ON-SITE SEWERAGE SYSTEMS

		DISPOSAL				
		Containment With Subsequent Off-Site Discharge	Conventional (Subsurface Soil)	ALTERNATIVE		
				Soil	Surface Water Discharge	Evapotranspiration (ET)
TREATMENT	NO TREATMENT	1. Holding Tanks	1. Cesspool Undesirable & Probably Unlawful	Undesirable	Unlawful	Undesirable
	CONVENTIONAL		1. Septic Tank-Soil Absorption Field	Septic Tank With: 1. Mound Systems 2. Underdrain Systems	Probably Unlawful	1. Septic Tank-Beds
	ALTERNATIVE	1. Water-Saving Fixtures	1. Alternative Treatment-Soil Absorption Field	Alternative Treatment With: 1. Mound Systems 2. Underdrain Systems	Alternative Treatment With: 1. Disinfection 2. Nutrient Removal	1. Alternative Treatment - ET Bed

Inspection--Due to the sensitive nature of the locations for on-site disposal systems, a mandatory on-site inspection is strongly recommended. This inspection would be the basis upon which the administrative agent would determine whether such a system is to be permitted for use at that location. Since the agent relies upon this inspection, it is further urged that the agent, himself, make all the inspections.

Licensing or certification of inspectors--As an alternative to the regulatory agency staff making all inspections, inspections could be made by agents, licensed or certified by the administrative authority. This licensure or certification is necessary to assure the quality of inspections. Such assurances can only be had if the inspectors license or certification is subject to suspension or revocation.

Education and/or licensing of installers--Due to the complex nature of some of these alternative disposal systems, a program to instruct the installers in the proper methods and techniques as well as the general design bases and characteristics of the alternative systems, is strongly recommended. Consideration should be given to licensing or certifying these installers to gain more assurance that the systems will be correctly installed.

Inspection checklists--A program to inspect the construction of these systems at each critical point during construction is strongly recommended. It is further recommended that this inspection be performed by the regulatory authority itself and that a thorough checklist be developed and used for all such inspections.

Inspection certification--If the regulatory authority cannot perform the inspection, it is recommended that the delegated agent chosen to perform such inspections be required to follow the same checklist and certify that the inspection was properly made. If this approval is selected in lieu of actual inspection by the regulatory authority, the regulatory authority must then provide a program to educate the inspectors in the proper design, construction and installation techniques of the alternate systems.

Suggestions Regarding Choice of Regulatory Authorities--

As stated earlier, it is believed that in many situations, the state is the best unit of government to administer a regulatory program to control on-site systems. Briefly, the bases for this conclusion are that: 1) the state government is less subject to pressure by individual citizens to issue or approve an on-site system for their particular property, 2) the state either has more expertise or has the resources to hire staff with specialized expertise in on-site systems, and 3) most states generally have the ultimate power to regulate any issue of state wide concern, i.e., proper application of on-site systems in order to protect public health and water quality.

The discussion regarding the choice of regulatory authorities which follows, suggests various levels of potential involvement by the state. In addition to regulation by the state, central management (and possibly ownership) by a local unit of government is also considered.

State review of all local government programs regulating alternative disposal systems--It is recommended that states adopt a mandatory plan review of all the alternative disposal systems approved by local units of government. This state review process conducted by the appropriate state authority prior to construction, would prevent the use of systems on improper sites by countermanding local approval whenever required. This could be accomplished by making the local unit of government's permit or plan approval subject to state level reversal within a given number of days.

To insure proper and timely inspections are performed, the state could require proof from the local governmental units that each alternative disposal system was inspected. This proof could be in the form of a statement certifying that the system was inspected during construction. The same type of state level review could be imposed upon a monitoring program to assure that the local unit of government was properly monitoring the systems after installation.

As an alternative to individual state review of each system, it is suggested that the state enact a mandatory review of the local regulatory programs. When a local program is found to be deficient, the state should impose a state program until the locality brings its program up to standards. The state would have to establish minimum standards for local programs, including enforcement practices, staff requirements, employment practices, siting and installation inspection requirements, etc. These standards could include design and siting requirements for on-site systems.

State regulatory program for all alternative disposal systems--If state level plan review is not desired, the state agency could seek complete responsibility for regulating these alternative systems. The basis for a state regulatory program would be the critical health and water quality problems which could result if the alternative systems were improperly regulated. Public health and water quality are both areas of state wide concern and would justify the pre-emption of local governmental control, even in states having strong local governments.

The appropriate state agency could perform some or all of the suggested regulatory techniques discussed above. Recognizing the importance of assuring proper evaluation of the sites where these alternative systems are to be located, the state agency, at a minimum, would want to perform the on-site inspection prior to approving the installation of a system. In addition, the plan review or permit process also would be performed by the state, since it is this process which supplies information which would be needed by the state to perform the on-site inspection. Other functions such as inspection of the installation could be performed by this same state agency. Clearly, the education and/or licensing of installers should be performed by the state.

Central management of non-central systems--A central management entity, such as special purpose district or other quasi-governmental unit, might be used to perform many of the regulatory functions described above. The ability of such districts to administer a regulatory program would depend both upon the enabling statutory language of the district and whether the local governmental units and state administrative authorities could delegate to or rely upon the districts to perform regulatory functions.

Briefly, better regulation of these alternative disposal systems might be obtained because the central management entity would be able to work closely with those persons within its jurisdiction and could perform some or all of the following functions:

1. Control the siting of each alternative system;

2. Control the design of each system;

3. Supervise and inspect the construction of each system;

4. Inspect and maintain each alternative system after installation;

5. Fund and staff a program to monitor the water quality within the district; and

6. Repair or replace the failing systems (charging the system owner the cost).

Alternative soil disposal system ownership--Finally, regarding regulatory authority, it is suggested that a unit of local government or centralized management entity could own all alternative soil disposal systems within its jurisdiction. Ownership would provide the regulatory authority with almost total control. The system would usually be located on the property of the individual served by the alternative system. Therefore, an easement would be required to give the governmental unit permission to install, maintain, remove and/or replace the system. The individual so served would be assessed the costs of providing these systems typically through a special assessment or user charge.

Important regulatory advantages would be achieved because the governmental unit could handle the design, installation, maintenance and operation of these alternative systems. As the owner, the governmental unit would have an incentive to assure that the systems were properly designed and installed. The individual, contrary to the typical situation today, would clearly have an incentive to report any malfunction or failure of the system since the system is owned by the governmental unit. This would especially be true if the costs of repair or replacement were not directly assessed against the homeowner at the time of failure but amortized through user charges. This would require that the unit of government would have the authority to own and operate such alternative on-site wastewater disposal systems or else enabling legislation would have to be sought.

Case History - Wisconsin's Program to Regulate Alternative Disposal Systems--

Description of the program--In June of 1975, the state agency responsible for the administration of on-site systems, the Wisconsin Department of Health and Social Services, Bureau of Environmental Health (hereinafter DHSS or department) began a two-year trial of a regulatory program which had been developed to control three types of alternative disposal systems--mound systems. Each county was given the option of determining whether it would participate in this trial of the regulatory program. Most counties chose to participate.

The regulatory program employed many (but not all) of the regulatory techniques discussed above. Most of the pre-approval control functions were performed by the state (DHSS) and the post-approval control functions were performed by the county authority responsible for regulation of on-site systems. These pre-approval regulatory techniques were as follows:

1. An individual application was required from the individual landowner for each proposed use requiring a mound system (Figure D-1),
2. The state reviewed each application,
3. An on-site inspection of the proposed location was conducted by either: a) state personnel (DHSS) or, b) a soil tester certified by the state (DHSS), and
4. A letter of approval was sent by the state (DHSS) to the applicant and the county regulatory official if the proposed site was found to be suitable (Figure D-2).

Most of the post-approval regulatory functions used were as follows:

1. County regulatory official was required to attend a state (DHSS) sponsored program for training regarding the inspection of the alternative systems,
2. The trained county official was required to inspect each alternative system during construction pursuant to a checklist provided by the state (DHSS) (Figure D-3),
3. The county official was required to certify that the inspection was properly performed,
4. This same official was required to return each checklist and the statement certifying that inspection was made to the state (DHSS),
5. The state (DHSS) issued a letter to the system owner authorizing use of the system if the checklist and certifying statement appeared to be in order (Figure D-4), and
6. The county government was required to have an appropriate official monitor each alternative system and file with the state (DHSS) a status report after one year's operation.

Thus, the state performed most of the pre-approval regulatory functions and the county and state split the post-approval regulatory functions.

Figure D-1. Application for Use of an Alternate System.

Plb. 108

WISCONSIN DEPARTMENT OF HEALTH & SOCIAL SERVICES
DIVISION OF HEALTH, BUREAU OF ENVIRONMENTAL HEALTH
P. O. BOX 309
MADISON, WISCONSIN 53701

APPLICATION FOR THE USE OF AN ALTERNATE SYSTEM

* *

Location ____________1/4______1/4______T______N, R______E (or) W

Town or Municipality __________________Street Address ______________________

Lot No. ___________, Block________, Subdivision____________County_________

Landowner's Name: ____

Mailing Address: ____

* *

I (We), the undersigned, hereby make application for permission to install an alternate system on the above-described premises. I recognize that the above premises are not suited for the conventional septic tank-soil absorption field and recognize that the alternate system applied for is to be used on my property which fails to meet the soil and site requirements of a conventional system. If permission is granted, I agree to have the system installed in conformance with the Division's approved plans and specifications. If the system is improperly installed, I agree to modify, repair or replace it if so ordered.

I further understand that the alternate system is more complex in nature than a conventional septic tank system and as such will require detailed inspection during construction and monitoring after the system is put into use. I agree to permit both county officials charged with administering county sanitary ordinances and Division employees or other authorized persons to have access to the above described premises at any reasonable time for the purpose of inspecting the construction or monitoring of the system. I further agree to either personally or by my agent contact the proper county official to arrange the time and date to begin construction of the system.

I understand that this application does not permit me (the applicant) or my agent (the contractor) to begin the installation of any alternate system. The Division or other authorized representative will perform an onsite inspection of the above-described premises. If the system is approved, the Division will send the applicant a Letter Authorizing the Construction of an Alternate System. I agree to permit Division employees or other authorized persons to have access to the premises at any reasonable time for the purpose of making such onsite evaluation and I further agree not to begin construction prior to the receipt of such a letter.

(continued)

Figure D-1 (continued)

-2-

I understand that this application does not permit me or any other person to discharge sewage into the alternate system, sought by this application, until I receive a Letter Approving the Use of an Alternate System from the Division. This letter will be sent by the Division after it receives, from the proper county officials, a checklist and statement certifying that the alternate system was properly constructed. I agree not to use or permit the use of the alternate system prior to receiving such a letter.

I recognize the limitations of the above-described premises and in consideration for the use of the alternate system applied for by this application, I agree to repair, modify or replace, at my expense, the alternate system if the county officials or the Division find the system to be malfunctioning. Further, I understand that the county or the Division may require that the alternate system be replaced with a holding tank or with a system of a more suitable design. I understand and agree that if a holding tank is required, I will have to make arrangements satisfactory to the Division for the disposal of the effluent.

I agree to give notice to any subsequent buyer that an application for an alternate system has been made and if installed, that the premises are served by an alternate system and further agree to give that buyer a copy of this application.

I understand that the Division and the county do not guarantee and do not provide a warranty (either implied or express) that the alternate system sought by this application will properly function. The Division receives this application subject to this understanding and subject to all the conditions and obligations set out in this application.

______________________________ ______________________________

Date Signature of Applicant

STATE OF WISCONSIN)
) ss.
COUNTY OF ________)

Subscribed and sworn to before me

this ______day of ________________, 19____.

____
Notary Public, State of Wisconsin

My Commission expires: ________________

Figure D-2. Alternative System Approval Letter

To: Property Owner

The Bureau of Environmental Health has reviewed plans, site survey information and installation details covering the construction of a (new or replacement) ______________ private sewage disposal system on your property located (location) ____ County, Wisconsin. The plans and installation details were prepared by name ____ title ______________ and received for approval on date ________________.

The site evaluation was conducted by name ________________ title ____________. The soil is name and/or description ______________. The soil percolation rate is ______________________________. The premises meets the soil and site requirements specificed for the use of Alternate System no. ____ which was developed by the University of Wisconsin Small Scale Waste Management Project.

The proposed system will serve a single family residence containing no. ____ bedrooms. The system has been sized in accord with the requirements set forth in the alternate system design criteria. Wastes from the home will discharge to a _____ gallon capacity septic tank which will discharge to a ______ gallon capacity pump chamber from which a pump having a capacity of ______ gallons per minute against a total dynamic head of _____ feet will discharge through ____ inch diameter pipe to the soil absorption system.

The proposed system is not in strict keeping with design criteria for private sewage disposal systems as established in the Wisconsin Administrative Code. However, certain features not specifically referred to in the "alternate designs" must be in accord with the regulations. Due to the existence of site soil limitations it is of utmost importance that the system be installed in complete accord with the plans and installation details and the conditions of approval contained herein; that the appropriate vil, co, twn, city official conduct thorough inspections at specified times, reporting his findings to this Division and that the contractor not deviate from this formal approval and follow directions or orders issued by the appropriate local or state authorities.

In accordance with Chapter 145, Wisconsin Statutes and Section H 62.24 (1), Wisconsin Administrative Code, approval to construct the alternate design private sewage disposal system is granted subject to the following conditions. If construction commences the Department will imply acceptance of the conditions.

(continued)

Figure D-2 (continued)

1. That the vil, co, twn, city authority administering the local sanitary ordinance permit the installation as proposed.
2. That a state septic tank permit be obtained.
3. That a copy of the plans and installation details covering the proposed system be supplied to the appropriate local officials.
4. That construction of the system not commence until the appropriate local official is notified and that construction proceed on a schedule dictated by that official.
5. That the owner of the system not commence its use nor permit any other person or persons to use it until a letter approving use is received from the Division.
6. That the appropriate local official be notified of the day when use of the system is to commence.
7. That a copy of this approval and the approved plans and installation details be kept on the premises during and after installation.
8. That appropriate local officials, employees of this Division and/or representatives of the University of Wisconsin Small Scale Waste Management Project be permitted to have access to the premises at any reasonable time for the purpose of inspecting and monitoring the system, including the conducting of any necessary bore holes or other physical examinations and the collection of samples of soil or liquids.
9. That in event the alternate design system or any of its component parts malfunction so as to create a health hazard by discharge of partially treated or untreated liquid wastes onto the ground surface or into the waters of the state the owner will repair, modify or replace at his expense (including the possibility of installation of a holding tank with proper disposal) the alternate design system with such action approved by the Division and the appropriate local official.
10. That any subsequent buyer of the premises be given notice that an alternate design system is installed and a copy of this letter of approval be given such buyer.
11. That the alternate design system installation be made in complete accord with the plans and installation details; appropriate sections of Chapter H 62, Wisconsin Administrative Code, that are not varied from in the alternate design; and the conditions of approval contained herein.

In granting this approval, the Division does not hold itself liable for any defects in construction; does not guarantee and further does not give warranty,

(continued)

Figure D-2 (continued)

either implied or express, that the system will function adequately or indefinitely; nor does it hold itself liable for any damages that may result from the installation of the system and reserves the right, after consultation with the University of Wisconsin Small Scale Waste Management Project and local personnel, to order changes or additions should conditions arise making such action necessary.

In case installation of the system has not actually commenced within two years from date, this approval is void. After two years, therefore, new application must be made for approval of these or other plans and installation details before any construction is undertaken.

By order of George H. Handy, M.D., State Health Officer.

H.E. Wirth, P.E., Director
Bureau of Environmental Health

Skk

cc: University of Wisconsin
Designer
County
District
Contractor if known and not designer
File

P & S to File
Owner
Designer

Figure D-3. Construction Checklist for the Inspection of Alternate Sewage Disposal Systems.

Construction Inspection of Alternate Design Sewage Disposal Systems

Wisconsin Department of Health & Social Services
Section of Plumbing & Fire Protection Systems

Plan Identification No. ____________

Installed for ____

A. Site Investigation at onset of construction

1. Name of Installer ____
2. County ______________ Inspector ______________ Date ________
3. Package # ______________
4. Preliminary onsite made by ____________________ Date ________
5. Depth to limiting factor (50% unconsolidated rock or estimated ground water level) ____________________
6. Percolation rate __________________
7. County installation permit number __________
8. Are percolation and soil boring holes evident? Yes ____ No ____
9. Is system located in area of soil tests? Yes ____ No ____
10. Is system located in area shown on state approved plans? Yes ____ No ____
11. Ground slope in area of system ____________________
12. Site data is correct as presented by C.S.T. and system designer? Yes ____ No ____

B. Inspection of Construction

1. Disposal site plowed and properly prepared? Yes ____ No ____
2. Disposal site conditions wet or damp? Wet_____Damp_____Dry_____
3. Type of fill material ______________________
4. Depth of fill (1' Minimum) ____________________
5. Is a crawler type tractor used? Yes ____ No ____
   a. Blade ____________ Bucket ____________
6. Has site been driven on by any vehicles? Yes ____ No ____

(continued)

Figure D-3 (continued)

7. Trench width as indicated on approved plans? Yes _____ No _____
8. Trench spacing as indicated on approved plans? Yes _____ No _____
9. Have trench bottoms been properly leveled? Yes _____ No _____
10. Trench length and number as shown on approved plans? Yes _____ No _____
11. Distribution piping proper diameter? Yes _____ No _____
12. Holes in distribution piping properly sized? Yes _____ No _____
13. Holes in distribution piping properly spaced? Yes _____ No _____
14. Holes in distribution piping in a straight line? Yes _____ No _____
15. Distribution holes drilled straight into piping? Yes _____ No _____
16. Depth of gravel below distribution piping ________________
17. Depth of gravel above distribution piping ________________
18. Thickness of marsh hay covering ____________________
19. Permanent marker at end of each trench _________________
20. Depth of fill over center of system ______________
21. Depth of fill over outer trenches _______________
22. Side slopes ___________________
23. Type of fill used above trenches __________________________
24. Depth of top soil ____________________
25. Seeded? Yes _____ No _____

C. Pumping Chamber

1. Diameter of inlet __________________
2. Diameter of outlet _________________
3. Head ___________________
4. Size of pump tank ____________________gallons
5. Draw down of gallons pumped per cycle ______________
6. Manufacturer and type of pump same as that indicated on approved plans? Yes _____ No _____
7. Quick disconnect provided? Yes _____ No _____
8. Diameter of manhole ________________
9. Height of manhole above finished grade ___________________
10. Diameter of vent ______________________
11. Height of vent above finished grade ____________________
12. Pump tank located as shown on approved plans? Yes _____ No _____

(continued)

Figure D-3 (continued)

D. Septic Tank

1. Properly installed? Yes _____ No _____

COMMENTS

I, the undersigned, hereby certify that the questions were answered on the basis of my personal inspection or knowledge of the construction of this alternate system and further that all data and answers recorded on this form are correct and to the best of my knowledge and belief.

Name: ______________________________ Signature: ____________________

Title: ______________________________

WE HAVE INCLUDED TWO COPIES OF THIS FORM FOR COMPLETION BY YOUR OFFICE. WHEN INSPECTION OF CONSTRUCTION IS COMPLETE, ONE COMPLETED FORM SHALL BE RETURNED TO THIS OFFICE WITHIN TEN (10) DAYS AFTER YOUR FINAL INSPECTION OF THIS ALTERNATE SYSTEM.

Date received by Section of Plumbing & Fire Protection Systems __________

Figure D-4. Letter of Approval for System Use

To: owner

Re: Approval for use of an alternate system

The Division has received from the county official responsible for enforcement of the sanitary ordinances in your county both a checklist and a statement certifying that the above described alternate system was inspected during construction. The Division has reviewed both documents and is satisfied that the above described alternate system was properly constructed.

The alternate system owner is reminded that this alternate system was necessary because of the unsuitability of the above described premises for a conventional septic tank-soil absorption field system. Due to the soil and site limitations of land like yours, alternate systems were developed to provide adequate treatment and safe disposal of wastewater. However, the Division does not guarantee and further, does not give a warranty (either implied or express) that your alternate system will function properly for any given period of time. Both when you applied for an alternate system and when you received a letter authorizing the construction of an alternate system, you agreed to accept the responsibility to repair or replace the above described alternate system if county officials or the Division find it to be malfunctioning. As the owner of this alternate system, you have further agreed that any repairs or replacement will be totally at your own expense with no recourse to the county or the Division for any part of the costs.

The alternate system may be used if the owner agrees to the following conditions:

1. That the system owner must permit both the county officials entrusted with the enforcement of county sanitary ordinances and employees of the Division or other authorized persons to have access to the above described premises at any reasonable time for the purpose of monitoring the alternate system;

2. That the system owner agrees to repair, modify or replace at his expense the above described alternate system if the county official or the Division finds the system to be malfunctioning. Any modification to or replacement of the alternate system must be acceptable to the Division;

3. That the owner agrees to give notice to the proper county officials and to the Division prior to beginning use of this alternate system;

4. That the owner agrees to inform any subsequent buyer that an alternate system has been installed by giving the buyer a copy of this letter, and a copy of the plans and specifications of the alternate system.

(continued)

Figure D-4 (continued)

By beginning to use this alternate system, the owner agrees to accept and to comply with all of the conditions set forth above. The Division will imply acceptance of these conditions if the owner begins use of this alternate system.

Sincerely,

James A. Sargent
Chief

JAS:skk

cc: District
County ZA
University of Wisconsin

Regulation of Surface Disposal Systems

The systems under consideration in this section are classified as alternative treatment - alternative disposal systems. While a wide range of alternative treatment methods are envisioned, only one type of alternative disposal is considered - surface discharge. It is both the alternative treatment methods as well as surface discharge which mandate a regulatory program more stringent than that used for conventional systems. This type of system differs from the other alternative disposal systems because of the discharge to a watercourse. Systems which rely on the soil for final disposal are seen as posing much less of an immediate threat if their respective treatment components fail. The lack of soil disposal, however, requires total reliance upon a properly functioning treatment component. Increased regulatory demands also are imposed because the treatment components require relatively more frequent maintenance when compared to the conventional septic tank. Thus the regulatory mechanism must be capable of meeting this increased responsibility. In addition, an institutional/regulatory mechanism must be available to monitor the discharge to assure that the on-site system is providing treatment sufficient to protect public health and water quality.

Unique Regulatory Aspects--

Of all the on-site systems, there are certain regulatory aspects which are unique to surface water discharge systems. These aspects are briefly noted below.

Applicable federal requirements--Unlike the other methods of on-site treatment and/or disposal, this method is regulated by federal law (PL 92-500). Any discharge to surface waters will have to comply with the federal requirements and state water quality standards adopted pursuant to PL 92-500. With perhaps some modification, an existing federal-state regulatory program would be used to control these systems.

Different state agencies involved--In many states there exists a dichotomy in regulation of sewerage. On-site systems are traditionally regulated by one state agency (typically that agency responsible for public health) while a different agency is responsible for water quality protection. However, since surface water discharging systems pose such a potential threat to water quality and public health, both state agencies may regulate these systems. The regulatory programs of two state agencies may pose some difficulties if their requirements may conflict. In any event, coordination between the two regulatory agencies will be necessary.

Federal Requirements Regulating Discharges to Water--

Off-lot discharging systems are regulated by the Federal Water Pollution Control Act Amendments of 1972, Section 402. Under this act the discharge of any pollutant into the nation's waters without a permit is unlawful. The permitting process (National Pollutant Discharge Elimination System - NPDES) is the keystone of the strategy for improving the quality of the nation's waters. A permit cannot be issued by the state water quality agency if the proposed discharge will violate any other provisions of the Act. Via this regulatory technique, the Act imposes upon all dischargers the requirement to meet effluent limitations, attain water quality related effluent limitations, meet water quality standards of the receiving waters, and meet total maximum daily load restrictions, if any.

The impact of this Act upon off-lot discharging systems for small flows is the same as upon any other discharger with the exception that some of the administrative requirements might be more burdensome to individual system owners.

To obtain a discharge permit, the proposed system would have to comply with all the provisions of the Act. Of primary importance is the effluent limitations. Other water quality requirements are all related to the water quality of the receiving waters. Therefore, until the receiving waters are identified, it is quite difficult to discuss the impact that these requirements would have.

The permit system and effluent limitations are discussed below. For illustrative purposes, a discussion of the Wisconsin NPDES permits and effluent limitations program is provided.

Pollutant discharge elimination permits--While this act granted the USEPA administrator the authority to issue discharge permits, it was clearly the intention of Congress that each state would be delegated authority to adopt and administer a permit program. Most states have undertaken such a permit program and, in most instances, additional state enabling legislation was required. For example, in Wisconsin, the state legislature enacted ch. 147, Wis. Stats, since the Department of Natural Resources lacked the necessary authority.

Regardless of whether the permit program is administered by a regulatory agency of the state or by the USEPA, the permit program imposes the same minimum requirements on point source dischargers of pollutants. Prior to approval of any state program, the appropriate USEPA regional administrator must determine that the designated state agency has adequate authority to administer a permit program which meets minimum standards.

A permit is required for the point source discharge of any pollutant or combination of pollutants, and can only be issued if the discharge will not violate the other provisions of the act. This means that surface water discharging systems would have to meet effluent limitations. If these systems are classified as publicly owned treatment works, they would have to meet effluent limitations based upon secondary treatment by July 1, 1977 (or any more stringent limitations necessary to meet water quality standards or other state and federal requirements). By July 1, 1983, publically owned treatment works are required to apply the best practicable waste treatment technology in order to obtain a discharge permit. The federal construction grant program is aimed at assisting publically owned treatment works to meet these objectives. If these systems are not publically owned treatment works, however, they must meet the effluent limitations of best available control technology by July 1, 1983 (Section 301, P.L. 92-500).

Both the discharges from publically owned treatment works and all other point source discharges may be further limited if the discharge would interfere with the attainment of specific water quality goals. In addition, permits may only be issued if the discharge will meet all the applicable standards of performance including maximum daily loads and toxic or pre-treatment effluent limitations.

The discharge permit program outlined in P.L. 92-500 specifies that a permit is required for on-site systems with surface water discharge and indicates that effluent limitations and other standards to be applied should not vary from state to state. Thus, the effect of the discharge permit program upon the development and regulation of on-site systems with surface water discharge should be uniform throughout the nation. Since P.L. 92-500 imposes only minimum requirements, a state could impose either more stringent effluent limitations or extend the scope of the discharges to cover land disposal.

The discharge permit program of a particular state should be examined to determine if it contains requirements in addition to those imposed by P.L. 92-500. In the discussion of Wisconsin's discharge permit program, it will be noted that groundwater quality protection is included within the permit program jurisdiction.

Wisconsin's pollutant discharge elimination system--Pursuant to P.L. 92-500, the Wisconsin legislature enacted ch. 147, Wis. Stats. to grant the DNR the needed authority to establish an approved pollutant discharge permit program (Sec. 147.01 (2), 1973).

Sec. 147.02 made the discharge of any pollutant into any waters of the state (surface and ground waters) unlawful unless such discharge is done under a permit issued by the department. No permits may be issued unless all applicable standards and effluent limitations are met. Sec. 147.025 mandated that within 180 days after enactment, every owner of any existing point source discharging pollutants into the waters of the state must apply for a permit. Also, every owner of a new point source wishing to commence discharging must apply for a permit at least 180 days prior to the date on which discharge is to commence.

On-site wastewater systems which propose to discharge any effluent into the surface waters of the state, therefore, would require a discharge permit. Further, due to the all encompassing definition of water, any on-site system discharging to a land disposal system could also be required to obtain a permit. However, the department has chosen not to exercise its statutory authority over on-site systems using land disposal. According to the rules adopted by the department, the discharge of domestic sewage to disposal system such as to septic tanks and drain fields is exempted.

By necessity, Wisconsin's discharge permit program parallels the underlying federal program since it separates publically owned treatment works from all other categories of point source discharges. The same effluent limitations are imposed upon each category by the Wisconsin programs as does the federal program. Similarly, the Wisconsin program prohibits the issuance of a permit unless the discharger meets all applicable performance, effluent, and water quality related standards, as well as total maximum daily loads (Sec. 147.02 (3) Wis. Stats., 1973). In summary, the Wisconsin program requires a permit for the lawful surface discharge of effluent from an on-site system which is issued only for surface discharging systems meeting the effluent limitations and other applicable standards.

Additional Possible State Regulatory Techniques--

Some regulatory techniques exist in other states which might apply to off-lot discharging systems. Two commonly used regulatory techniques are discussed below in regard to their impact or applicability to individual off-lot discharging systems. While not every state currently employs each of these, the techniques have been basic to the regulation of publically owned treatment works and may apply to small wastewater flow systems, as well. Wisconsin's program is used as an illustration to aid in determining whether these regulatory techniques should be applied to small wastewater flow systems.

Plan approval--Most states and some local units of government require the owner of a wastewater facility to submit plans prior to construction of the proposed system. Typically, this plan review is performed by the state agency having responsibility for water quality protection. Publically owned treatment works usually receive 75 percent federally of the costs of construction through federal construction grants. Therefore, the state's plan review is done in compliance with federal regulations followed by a review by the appropriate EPA regional office. Plan review and approval by the state agency and EPA regional office is certain to be a requirement to be imposed on on-site systems where federal construction assistance is sought.

Depending on ownership, on-site systems proposing surface water discharge may be eligible for federal construction grants and thus, be reviewed under the federal grants procedure. For those facilities not eligible for federal assistance, the state or local governmental unit may or may not have a plan review and approval process which would apply to these systems. The "non-federal grants" portion of any state's plan review and approval requirements must be examined to determine whether on-site systems are included within the definition of the systems which are required to obtain plan approval. Typically, these state statutes or department rules may require review and approval of the plans for a proposed "wastewater treatment works or facility." There is no uniform state approach to consideration of proposed individual on-site systems with surface water discharge as a "works or facility" requiring plan review.

A potential problem may arise in states where individual on-site-surface water discharging systems have not been used previously. This occurs because they are very similar to the conventional wastewater treatment facility in that they discharge to the surface waters. Thus, the state agency which has the responsibility for regulation of sewerage facilities must determine whether on-site-surface water discharging systems are to be regulated as conventional sewerage facilities. If the agency determines that the systems are under its jurisdiction, then the agency must impose its requirements upon the on-site systems. However, the outcome of this decision may not be critical because individual surface discharging systems not characterized as treatment works (and, thus not regulated by the agency responsible for sewerage systems) are probably regulated by the state and/or local agency which regulates individual on-site systems. With regulation practically assured, the only question is what form the regulation will take.

Wisconsin statutes and administrative rules are examined as an illustration of on-site-surfacing discharge systems characterized as sewerage facilities subject to the plan review and approval. State statutes (Sec. 144.04 Wis. Stats., 1973) require that every owner file a certified copy of the complete plans of the proposed "system or plant" or extension thereof and other information concerning maintenance, operation and other details as may be required by the Department of Natural Resources (DNR), the state agency primarily responsible for water quality. The DNR has 90 days to approve, conditionally approve or reject the plans. An owner who fails to comply with this requirement before installing a system or plant would be subject to a fine of from $10 to $5,000. Each day of continuing violation constitutes a separate offense (Sec. 144.57 Wis. Stats., 1973).

The phrase "system or plant" is defined by statute to include "water and sewerage systems and sewerage and refuse disposal plants" (Sec. 144.01 (6) Wis. Stats., 1973). The term "sewerage system" is also defined by statute to mean (Sec. 144.01 (5) Wis. Stats., 1973):

> ". . .all structures, conduits and pipe lines by which sewage is collected and disposed of, except plumbing inside and in connection with buildings served, and service pipes from building to street main."

It is clear that any type of on-site treatment and disposal wastewater system could be included within the definition of "system or plant." Thus, the requirements of plan submission and department review and approval would apply to these surface discharging on-site systems. Current practice at the department, however, does not require the submission of plans, departmental review or approval for a single on-site system using soil disposal of the effluent, regardless of the number of persons served by the system, although the department has just recently imposed the plan review and approval requirement upon a community system consisting of individual septic tanks - small diameter gravity sewer pipe discharging into common soil disposal fields. It may be inferred that if an on-site system were proposed to serve a single family by treating and then discharging effluent to the surface waters of the state, the department might impose the plan review and approval requirement.

The requirement that owners of each on-site-surface water discharging system submit plans and delay construction of the system until receiving departmental approval may impose increased costs upon the owner. First, the direct cost of plan preparation may increase. In Wisconsin, conventional on-site systems serving one or two family residences must comply only with a minimum state standard and are not required to obtain any plan approval from the state agency (Department of Health and Social Services - DHSS) which regulates all conventional on-site systems.

Other less direct costs may be incurred during the plan review process, which may take as long as 90 days. Thus, the owner would either have to submit plans well in advance of his construction date or subject himself to the risks of construction delays with all of the attendant costs.

In conclusion, surface water discharging systems in Wisconsin easily could be characterized as a "sewerage system" and thereby be subject to plan review and approval by the state water quality agency. Regardless of this characterization, the owner of the proposed system would still be required to comply with all local regulatory procedures as well as any that might be imposed by the Wisconsin Department of Health and Social Services.

Certification of sewerage treatment plant operators--Most states and some local units of government have a program to examine and certify sewerage treatment plant operators. Typically, those states which have such a program require that only certified operators may be employed to operate certain types of treatment facilities. This requirement is just as important as the certification program since, without this requirement, many owners of treatment facilities probably would not employ state certified operators because a premium might have to be paid to obtain them. Therefore, to assure trained certified operators are employed, a state program should consist of both the certification process and the requirement that only certified operators may operate sewerage facilities within the state.

The desired goal of any certification program is to have qualified operators operating all of the treatment facilities within each state. This certification procedure is to assure the capability of the wastewater facility operators. Although other devices or techniques could be used to assure the availability of qualified operators, this certification program is perhaps the easiest to administer.

The certification program, or other techniques or devices to accomplish this goal, can be effectively implemented by including the use of a certified operator as a condition of the discharge permit. This approach may require enabling legislation in some states. Further, those states which require plan approval may also make their approval conditional upon the assurance that only certified operators will operate the treatment facility.

While it is clear that the treatment works serving large metropolitan areas should be operated only by qualified operators, the issue becomes less clear as the communities served decrease in size. That is, will the state agency responsible for administering the certification program determine that on-site systems employing surface water discharge, but serving only small populations, such as a single family, require a certified operator. This determination should depend on a thoughtful consideration of the characteristics of each on-site system. After review, the appropriate state agency could determine whether a certified operator would be required. This requirement would be imposed for one or more of the following reasons:

1. The system is sensitive and easily upset requiring close monitoring.

2. The system is complex, or

3. The system is likely to pose a threat to public health or water quality.

While this list of reasons is by no means complete, it is offered to aid in the judgement of whether a state will require a certified operator for various types of on-site systems. This is a question of how the on-site system will be viewed. If viewed as being analogous to a municipal treatment facility, the state will likely require a certified operator. Because of public health and water quality concerns, it is anticipated that on-site systems which propose surface water discharge also will be required to have a certified operator. It is necessary to examine the relevant statutes, rules and guidelines in the state of interest to determine whether that state has a certified operator program and whether the requirement includes on-site systems.

Because of the varying complexity of treatment facilities, it would be expected that states would classify the facilities in order of complexity. Certification of operators should reflect these classes. For example, it would be unnecessary for the operator of a small stabilization lagoon to be qualified to operate a large metropolitan activated sludge treatment plant. It is anticipated that most states which have a certification program have also defined classes of treatment facilities with appropriate minimum levels of training, education (degree) and operating experience for the operators of each treatment facility class.

If included under a state operator certification program, on-site systems including those with surface discharge, probably would be classified in the least complex class of treatment facilities. The amount of training, education and operating experiences which the opeartor would be required to have therefore, would be the least demanding of the levels required for certification. However, it is not known whether the requirement for a certified operator of on-site systems will pose an unreasonable burden for the system owner. Of course, the owner could become certified to operate his own on-site system, assuming that he could meet the basic minimum requirements for certification.

Wisconsin Statutes require that all treatment facilities be operated by certified operators. Pursuant to this requirement, administrative rules were adopted to classify both treatment facilities and corresponding operators. The statutes provide that the Department of Natural Resources shall establish an examination program for the certification of sewage treatment plant operators and impose a deadline after which no person shall operate a sewage treatment plant unless he is certified (Sec. 144.025 (2)(1) Wis. Stats., 1973). According to the administrative rules adopted pursuant to this statute, a sewage treatment plant is defined as (Sec. NR 114.02 (3), Wis. Admin. Code, 1971):

> ". . . any facility or group of units provided for the treatment of sanitary sewage and/or industrial waste."

The definition specifically excludes septic tank and soil disposal systems. By implication, other on-site systems, especially those with surface water discharge, are included and are subject to the certified operator requirement.

A further examination of these rules shows that the Department has classified sewage treatment plants based on one of two criteria: flow or complexity of the process employed by the plant. Individual domestic on-site systems are not included either under the lowest flow (.1 MGD) criterion nor the complexity criterion. However, an additional classification, Class IV which includes "All other plants which treat sanitary sewage," would include on-site systems (Sec. NR 114.08 (1)(d) Wis. Admin. Code, 1973). These rules establish 5 grades of sewage treatment plant operators and require that an individual to become certified in any grade must meet one of the combination of educational and experience requirements and pass the appropriate examination (Sec. NR 114.09 (1) Wis. Admin. Code, 1973). The rules also provide that anyone holding a grade I, II, III or IV certificate may operate a class IV sewage treatment plant (the probable class for on-site-surface discharge systems). Thus, a grade IV operator would be able to operate on-site systems. The rules define grade IV experience requirements to be (Sec. NR 114.09 (d) Wis. Admin. Code, 1973):

> "Completion of special course of training in sewage treatment and demonstration of aptitude in operation of sewage treatment works."

These special courses are offered one or more times per year and might be one to two weeks in length.

Suggested Regulatory Techniques--

Since each on-site surface water discharge probably will have to comply with federal (PL 92-500) and state water quality standards, the regulation of these systems should be assured. As discussed previously, each discharge will be required to obtain a pollutant discharge permit (NPDES) and meet federally set discharge standards (currently secondary treatment requirements). Also, the federal requirements include monitoring and reporting minimums which must be met by each permittee. Therefore, the regulatory program for this category of treatment-disposal will be largely determined by the federal requirements under P.L. 92-500. However, the state or local regulatory agency may employ additional regulatory techniques as suggested below.

Certified treatment plant operators--The requirement that certified operators be employed to operate and maintain on-site surface water discharging wastewater systems already may be imposed by state law or local government ordinance. If not, it is recommended that consideration be given to imposing such a requirement. It is felt that certification of treatment plant operators would provide a regulatory technique which could significantly strengthen the existing federal-state regulatory program.

Typically, one or more agencies of state government will have the responsibility for regulating these surface discharge systems. Therefore, it is recommended that certification be imposed at the state level. The exact mechanisms and the designation of the appropriate state agency are not considered here. However, the program must be rigorous enough to guarantee that only properly trained operators would be certified. If certified operators are required, the state agency which regulates on-site systems should gain increased assurance that the systems will be adequately operated.

While it might appear that requiring a certified operator would be burdensome for the system owner, there are several methods which could reduce this burden.

In those states which require a certified operator for some or all types of on-site systems (especially surface water discharge), some certified operators will contract their services to system owners. Since the operation of these individual systems is not time intensive, the certified operator could service many systems. Depending upon the size of the market in any given locale, one or more certified operators might be able to earn an adequate income by supplying such services.

Alternatively, it is possible for several system owners to hire a single certified operator to operate all of their systems. This approach could be undertaken either through an informal agreement between owners, or through the special district approach.

Plan review by state or local government--The extensive plan review required for public treatment works may be applicable to on-site systems using surface water discharge. However, most regulatory programs which control the conventional on-site system contain a plan review requirement, albeit somewhat rudimentary when compared to the requirements of public treatment works. It is recommended that the regulatory program contain adequate authority to require the potential on-site system owner to submit detailed plans for review. In most jurisdictions, this requirement already exists or the rules and regulations can be easily modified to require expanded review when surface water discharge is sought.

Note that adequate plan review will be obtained via the facilities planning or procedures imposed by P.L. 92-500 whenever the proposed treatment work is to receive a federal construction grant. Although on-site systems are grant eligible, it is recognized that state funding lists generally assign low priority to small projects. Rather than waiting several years until funds are available, some owners may decide to proceed without federal assistance. In such cases, relying on the grant review process alone will be an insufficient control.

Central management of non-central systems--Adequate operation and maintenance of these alternative treatment-surface discharge systems could be provided by a central management entity such as a general or special purpose unit of government. It is strongly recommended that states and/or local units of government, impose, before granting approval to the use of on-site surface water discharging systems, the requirement that the systems are to be operated and maintained (and perhaps also owned) by a central management entity.

Governmental ownership--Where permitted by state constitution or statutes, ownership by the state or local units of government provides the best assurance of proper operation and maintenance of these alternative treatment-surface discharge systems. Other advantages were discussed previously and will not be repeated here.

LAND USE IMPLICATIONS OF ALTERNATIVE ON-SITE WASTEWATER DISPOSAL SYSTEMS

In many areas of the country, planners have relied upon two related facts concerning wastewater service in planning ex-urban and other low density areas. The first of these is the unsuitability of some soils within their jurisdiction for conventional septic tank systems. The second fact is that public sewerage systems are too expensive, in many cases, for less densely populated areas. Unfortunately, in areas unsuitable for septic tank systems, the only other alternative usually considered is a public central sewer system. However, expense frequently rules out its use, discouraging development of the area. New technology, either in the form of alternative disposal systems or more cost effective methods of public sewerage could make development of these areas more practical. Communities which previously discouraged development in areas where septic tank systems were not feasible, may want to reevaluate their land use plans and regulations.

Alternative Disposal Systems -- Case Studies of Potential Impact

The alternative disposal systems discussed in the body of this report provide methods for safe treatment and disposal of small wastewater flows on sites previously considered unsuitable. Specifically, mound disposal systems for very slowly permeable soils and for permeable soils over shallow creviced or porous bedrock have already been developed. Work continues on the development of alternative disposal systems which are not dependent on soil or site conditions, i.e., surface water discharging systems.

The implications of these existing and potential alternatives pose for land use is obvious, especially when one considers that an estimated 68 percent of the United States has been judged to be unsuitable for the conventional septic tank system (Wenk, 1971).

Of course, the unstated premise here is that sanitary ordinances and subdivision regulations are sufficiently enforced to curtail or prohibit the installation of conventional septic tank systems on unsuitable sites. Strict enforcement of septic tank system siting requirements is often the exception rather than the rule in many areas, however.

Two case studies suggest that the development of alternate systems could have considerable impact, especially in those areas where development with septic tank systems is prevented because of unsuitable soils (Amato and Goehring, 1974; Water Resources Management Workshop, 1973). The alternate systems are designed to provide safe, effective disposal of domestic wastewater for certain areas that had been unsuited for septic tank systems. Thus, such systems are available for use in any area which meets the reduced site and soil requirements imposed by the alternatives. With this environmental limitation removed, only other land use controls, if any, will limit the developability of that area.

Obviously, the potential impact on a given governmental unit increases as the amount of previously unsuitable land becomes developable through the use of alternate systems. Those governmental units relying on septic system siting criteria should be aware of the development of new alternate methods of wastewater disposal. Those areas which have restricted development based on the unsuitability of lands for conventional systems may need to adopt more sophisticated land use plan and control systems. The development of environmental resource data should be a top priority assignment of planning commissions so they will be in a better position to advise local officials concerning land use decisions.

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APPENDIX E

GLOSSARY

A horizon: An horizon formed at or near the surface, but within the mineral soil, having properties that reflect the influence of accumulating organic matter or eluviation, alone or in combination.

AB horizon: A transitional horizon between the A and B horizons, having features of the A horizon in its upper part and features of the B horizon in its lower part, but without a clearly defined point to indicate where these features separate.

absorption: The process by which one substance is taken into and included within another substance, as the absorption of water by soil or nutrients by plants.

activated sludge: Sludge floc produced in raw or settled wastewater by the growth of zoogleal bacteria and other organisms in the presence of dissolved oxygen and accumulated in sufficient concentration by returning floc previously formed.

activated sludge process: A biological wastewater treatment process in which a mixture of wastewater and activated sludge is agitated and aerated. The activated sludge is subsequently separated from the treated wastewater (mixed liquor) by sedimentation and wasted or returned to the process as needed.

adsorption: The increased concentration of molecules or ions at a surface, including exchangeable cations and anions on soil particles.

aeration: The bringing about of intimate contact between air and a liquid by one of the following methods: spraying the liquid in the air; or by agitation of the liquid to promote absorption of air.

aeration tank: A tank in which sludge, wastewater, or other liquid is aerated.

aerobic: (1) Having molecular oxygen as a part of the environment. (2) Growing or occurring only in the presence of molecular oxygen, as aerobic organisms.

aerobic bacteria: Bacteria that require free elemental oxygen for their growth.

agar: A polysaccharide obtained from various species of seaweeds, used to gel materials (especially microbiological media). It cannot be broken down by most bacteria.

aggregate, soil: A group of soil particles cohering so as to behave mechanically as a unit.

aggregation: The act or process of forming aggregates, or the state of being aggregated.

air-dry: The state of dryness of a soil at equilibrium with the moisture contained in the surrounding atmosphere.

Alfisol: An order of soils having a B2 horizon high in crystalline clay and a moderately high level of exchangeable bases. These soils usually occur in the climatic range associated with scrub to well-developed deciduous forests.

Alluvial soil: (1) A soil developing from recently deposited alluvium and exhibiting essentially no horizon development. (2) A great soil group of the azonal order.

alluvium: Sediment deposited on land from streams.

alternative disposal: Any disposal method other than conventional subsurface soil disposal.

alternative treatment: Any treatment method other than conventional septic tank treatment.

ameba, p. amebae: A unicellular organism with an indefinite changeable form. Also spelled amoeba.

amino acid: An organic compound containing both a carboxyl ($COOH$) and an amino ($-NH_2$) group, bonded to the same carbon atom. The 20 amino acids which are the subunits of proteins vary in the structure of their side chains.

ammonification: The biochemical process whereby ammoniacal nitrogen is released from nitrogen-containing organic compounds.

anaerobic: (1) The absence of molecular oxygen. (2) Growing in the absence of molecular oxygen (such as anaerobic bacteria).

anaerobic bacteria: Bacteria that grow only in the absence of free elemental oxygen.

anaerobic contact process: An anaerobic waste treatment process in which the microorganisms responsible for waste stabilization are removed from the treated effluent stream by sedimentation or other means and held in or returned to the process to enhance the rate of treatment.

anaerobic digestion: the degradation of organic matter brought about through the action of microorganisms in the absence of element oxygen.

anaerobic respiration: The metabolic process in which electrons are transferred from one compound to an inorganic acceptor molecule other than oxygen. The most common acceptor molecules are carbonate, sulfate, and nitrate.

anaerobic waste treatment: waste stabilization brought about through the action of microorganisms in the absence of air or elemental oxygen. Usually refers to waste treatment by methane fermentation.

antibiotic: A chemical substance produced by certain molds and bacteria which inhibits the growth or kills other microorganisms. Some antibiotics can now be synthesized chemically and others can be altered by chemical methods so as to enhance their usefulness.

antibody: Protein produced by the body in response to the presence of an antigen; can combine specifically with that antigen.

antigen: A substance that can incite the production of specific antibodies and can combine with those antibodies.

attenuated: Modified so as to be incapable of causing disease under ordinary circumstances.

autoclave: A device employing steam under pressure and used for sterilizing materials stable to heat and moisture.

autotroph: An organism that can utilize CO_2 as its main source of carbon.

autotrophic: Capable of utilizing carbon dioxide or carbonates as the sole source of carbon and obtaining energy for the reduction of carbon and biosynthetic processes from radiant energy (photoautotroph) or oxidation of inorganic substances (chemoautotroph).

B horizon: An horizon immediately beneath the A horizon characterized by a higher colloid (clay or humus) content, or by a darker or brighter color than the soil immediately above or below, the color usually being associated with the colloidal materials. The colloids may be of illuvial origin, as clay or humus, they may have been formed in place (clays, including sesquioxides), or they may have been derived from a texturally layered parent material.

backwashing: the operation of cleaning a filter by reversing the flow of liquid through it and washing out matter previously captured in it. Filters would include true filters such as sand and diatomaceous-earth types but not other treatment units such as trickling filters.

bacteria: Primitive plants, generally free of pigment, which reproduce by dividing in one, two or three planes. They occur as single cells, groups, chains or filaments, and do not require light for their life processes. They may be grown by special culturing out of their native habitat.

aerobic: Bacteria which require free (elementary) oxygen for their growth.

anaerobic: Bacteria which grow in the absence of free oxygen and derive oxygen from breaking down complex substances.

bacteriophage: A virus that infects bacteria; often abbreviated "phage."

bar: A unit of pressure equal to one million dynes per square centimeter, which is nearly equal to the standard atmosphere.

base-saturation percentage: The extent to which the adsorption complex of a soil is saturated with exchangeable cations other than hydrogen or aluminum. It is expressed as a percentage of the total cation-exchange capacity.

bedrock: The solid rock underlying soils and the regolith at depths ranging from zero (where exposed by erosion) to several hundred feet.

biochemical oxygen demand (BOD): The amount of oxygen required to maintain aerobic conditions during decomposition.

biomass: The total mass of living organisms in a given volume (for example, in seawater it is the total mass of living organisms per liter). The total weight of all organisms in any particular environment.

black water: Liquid and solid human body waste and the carriage waters generated through toilet usage.

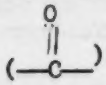
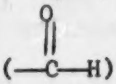
bulk density, soil: The mass of dry soil per unit bulk volume. The bulk volume is determined before drying to constant weight at 105° C.

C horizon: Horizon that normally lies beneath the B horizon but may lie beneath the A horizon where the only significant change caused by soil development is an increase in organic matter, which produces an A horizon. In concept, the C horizon is unaltered or slightly altered parent material.

calcareous soil: Soil containing sufficient calcium carbonate (often with magnesium carbonate) to effervesce visibly when treated with cold 0.1N hydrochloric acid.

capillary attraction: A liquid's movement over or retention by a solid surface due to the interaction of adhesive and cohesive forces.

capillary fringe: A zone just above the water table that is maintained in an essentially saturated state by capillary forces of lift.

carbohydrate: An organic compound consisting of many hydroxyl (—OH) groups and containing either a ketone  $(-\overset{\overset{O}{\|}}{C}-)$ or aldehyde  $(-\overset{\overset{O}{\|}}{C}-H)$ group.

Examples include sugars, cellulose, glycogen, starch.

carrier: An individual who has pathogenic microbes in or on his or her body without showing any signs of illness. The carrier state occurs during incubation and convalescence of infectious disease and with asymptomatic infection, colonization, or contamination. As usually used, the term implies that the microbes have access to the exterior of the body and thus potentially to other people.

catalase: An enzyme, found in human beings and many microorganisms, which degrades hydrogen peroxide to oxygen and water.

cation exchange: The interchange between a cation in solution and another cation on the surface of any surface-active material such as clay or organic colloids.

cation-exchange capacity: The sum total of exchangeable cations that a soil can adsorb. Sometimes called total-exchange capacity, base-exchange capacity, or cation-adsorption capacity. Expressed in milliequivalents per 100 grams or per gram of soil (or of other exchanges such as clay).

cellulose: A polysaccharide, composed of glucose subunits. The most abundant organic compound in the world.

chemical oxygen demand (COD): A measure of the oxygen equivalent of that portion of organic matter that is susceptible to oxidization by a strong chemical oxidizing agent.

chemical precipitation: The addition to sewage of such chemicals as will, by reaction with one another and the constituents of the sewage, produce a flocculent precipitate and thus hasten sedimentation.

chlorination: Treatment with chlorine or bleaching powder for the purpose of disinfection, the retardation of decomposition or the oxidation of organic matter.

chlorine residual: The total amount of chlorine (combined and free available chlorine) remaining in water, sewage or industrial wastes at the end of a specified contact period following chlorination.

clarifiers: Settling tanks; purpose of a clarifier is to remove settleable solids by plain gravity or colloidal solids by coagulation following chemical flocculation; will also remove floating oil and scum through skimming.

clay: (1) A soil separate consisting of particles < 0.002 mm in equivalent diameter. (2) A textural class.

clay mineral: (1) Naturally occurring inorganic crystalline or amorphous material found in soils and other earthy deposits, the particles being predominantly < 0.002 mm in diameter. Largely of secondary origin.

cleavage: Tendency to break in the same direction, thus yielding fragments of predictable shape.

clod: An artificially produced, compact, coherent mass of soil ranging in size from 5 or 10 mm to as much as 8 or 10 inches.

columnar structure: A soil structural type with a vertical axis much longer than the horizontal axes and a distinctly rounded upper surface. See prismatic structure.

coagulation: In water and wastewater treatment, the destabilization and initial aggregation of colloidal and finely divided suspended matter by the addition of a floc-forming chemical or by biological processes.

coarse texture: The texture exhibited by sands, loamy sands, and sandy loams except very fine sandy loam.

coliform-group bacteria: A group of bacteria predominantly inhabiting the intestines of man or animal, but also occasionally found elsewhere. It includes all aerobic and facultative anaerobic, Gram-negative, non-spore-forming bacilli that ferment lactose with production of gas. Also included are all bacteria that produce a dark, purplish-green colony with metallic sheen by the membrane-filter technique used for coliform identification. The two groups are not always identical, but they are generally of equal sanitary significance.

colloids: The finely divided suspended matter which will not settle and the apparently dissolved matter which may be transformed into suspended matter by contact with solid surfaces or precipitated by chemical treatment. Substances which are soluble as judged by ordinary physical tests, but will not pass through a parchment membrane.

concentration: The relative content of a substance. Example: The strength of a solution.

conductivity, hydraulic: As applied to soils--the ability of the soil to transmit water in liquid form through pores.

consistence: (1) The resistance of a material to deformation or rupture. (2) The degree of cohesion or adhesion of the soil mass. Terms used for describing consistence at various soil moisture contents are:

wet soil: Non sticky, slightly sticky, sticky, very sticky, nonplastic, slightly plastic, plastic, and very plastic.

moist soil: Loose, very friable, friable, firm, very firm, and extremely firm.

dry soil: Loose, soft, slightly hard, hard, very hard, and extremely hard.

cementation: weakly cemented, strongly cemented, and indurated.

conventional disposal: The method of disposal which relies on subsurface soil infiltration of the treated wastewater.

conventional treatment: Treatment which is effected by using a septic tank as it is generally used to treat wastewater.

crumb: A soft, porous, more or less rounded ped from 1 to 5 mm in diameter.

crust: A surface layer on soils, ranging in thickness from a few millimeters to perhaps as much as an inch, that is much more compact, hard, and brittle, when dry, than the material immediately beneath it.

cytopathic effects (CPE): Observable changes in cells in vitro produced by viral action; for example, lysis of cells or fusion of cells.

deflocculate: To separate the individual components of compound particles by chemical and/or physical means. See disperse.

degradation: The breakdown of substances by biological action.

denitrification: The biochemical reduction of nitrate or nitrite to gaseous molecular nitrogen or an oxide of nitrogen.

digestion: (1) The biological decomposition of organic matter in sludge, resulting in partial gasification, liquefaction and mineralization. (2) The process carried out in a digester.

disease: A process resulting in tissue damage or alteration of function, producing symptoms, or noticeable by laboratory or physical examination.

disinfection: Killing pathogenic microbes on or in a material without necessarily sterilizing it. Use of this term usually implies that a liquid or gaseous chemical agent is employed for the microbial killing.

disperse: (1) To break up compound particles, such as aggregates, into the individual component particles. (2) To distribute or suspend fine particles, such as clay, in or throughout a dispersion medium, such as water.

dissolved oxygen: The oxygen dissolved in water, wastewater or other liquid, usually expressed in milligrams per liter (mg/L), parts per million (ppm) or percent of saturation. Abbreviated DO.

dissolved solids: Theoretically, the anhydrous residues of the dissolved constituents in water. Actually, the term is defined by the method used in determination. In water and wastewater treatment the standard methods tests are used.

double layer: In colloid chemistry, a double layer of electrical charges, one consisting of the charges provided by the solid phase (usually negative) and the second by adsorbed ions of opposite charge.

E. coli: Abbreviation of Escherichia coli.

effluent: Sewage, water or other liquid, partially or completely treated or in its natural state, as the case may be, flowing out of a reservoir, basin, treatment plant or part thereof.

effluent weir: A weir at the outflow end of a sedimentation basin or other hydraulic structure.

electron: A subatomic particle of negative electrical charge that orbits the positively charged nucleus of an atom. For maximum stability an atom must have a certain number of electrons in its outermost orbit.

eluviation: The removal of soil material in suspension from a layer or layers of a soil. (Usually, the loss of material in solution is described by the term leaching).

enzyme: An organic catalyst. A protein molecule which lowers the activation energy of substrates allowing them to react at temperatures compatible with life.

epidermis: The outermost skin layers.

Escherichia coli (E. coli): One of the species of bacteria in the coliform group. Its presence is considered indicative of fresh fecal contamination.

eutrophic: A term applied to water that has a concentration of nutrients optimal, or nearly so, for plant or animal growth.

evapotranspiration: The combined loss of water from a given area, and during a specified period of time, by evaporation from the soil surface and by transpiration from plants.

extended aeration: A modification of the activated sludge process which provides for aerobic sludge digestion within the aeration system. The concept envisages the stabilization of organic matter under aerobic conditions and disposal of the end products into the air as gases and with the plant effluent as finely divided suspended matter and soluble matter.

facultative anaerobic bacteria: Bacteria which can adapt themselves to growth in the presence, as well as in the absence, of oxygen. May be referred to as facultative bacteria.

fats: Simple lipids consisting of esters of glycerol with fatty acids.

fermentation: The metabolic process in which the final electron acceptor is an organic compound.

field capacity: The water remaining in a field soil that has been thoroughly wetted and drained until free drainage has practically ceased.

filter: A device or structure for removing solid or colloidal material, usually of a type that cannot be removed by sedimentation, from water, wastewater or other liquid. The liquid is passed through a filtering medium, usually a granular material but sometimes finely woven cloth, unglazed porcelain, or specially prepared paper. There are many types of filters used in water or wastewater treatment.

filtering medium: Any material through which water, wastewater or other liquid is passed for the purpose of purification, treatment or conditioning.

filter clogging: The effect occurring when fine particles fill the voids of a sand filter or biological bed or when growths form surface mats that retard the normal passage of liquid through the filter.

filtrate: The liquid which has passed through a filter.

final effluent: The effluent from the final treatment unit of a wastewater treatment plant.

final sedimentation: The separation of solids from wastewater in a final settling tank.

final settling tank: A tank through which the effluent from a trickling filter or an aeration or contact-aeration tank is passed to remove the settleable solids. Also called final settling basin.

fine texture: The texture exhibited by soils having clay as a part of their textural class name.

first-stage biochemical oxygen demand: That part of oxygen demand associated with biochemical oxidation of carbonaceous, as distinct from nitrogenous, material. Usually, the greater part, if not all, of the carbonaceous material is oxidized before the second stage, or substantial oxidation of the nitrogenous material, takes place. Nearly always, at least a portion of the carbonaceous material is oxidized before oxidation of nitrogenous material even starts.

five-day BOD: That part of oxygen demand associated with biochemical oxidation of carbonaceous, as distinct from nitrogeneous, material. It is determined by allowing biochemical oxidation to proceed, under conditions specified in Standard Methods, for 5 days.

floc: Small gelatinous masses formed in a liquid by the reaction of a coagulant added thereto, through biochemical processes or by agglomeration.

flood plain: Flat or nearly flat land on the floor of a river valley that is covered by water during floods.

fragipan: A natural subsurface horizon with high bulk density relative to the solum above, seemingly cemented when dry, but when moist showing a moderate to weak brittleness. The layer is low in organic matter, mottled, slowly or very slowly permeable to water, and usually shows occasional or frequent bleached cracks forming polygons. It may be found in profiles of either cultivated or virgin soils but not in calcareous material.

gravitational potential: See potential, soil water.

grease trap: A device by means of which the grease content of sewage is cooled and congealed so that it may be skimmed from the surface.

grey water: Liquid and solid wastes generated through usage of water-using fixtures and appliances excluding the toilet and possibly the garbage disposal.

groundwater: That portion of the total precipitation which at any particular time is either passing through or standing in the soil and the underlying strata and is free to move under the influence of gravity.

hardpan: A hardened soil layer, in the lower A or in the B horizon, caused by cementation of soil particles with organic matter or with materials such as silica, sesquioxides, or calcium carbonate. The hardness does not change appreciably with changes in moisture content and pieces of the hard layer do not slake in water. See caliche and claypan.

head: The energy, either kinetic or potential, possessed by each unit weight of a liquid, expressed as the vertical height through which a unit weight would have to fall to release the average energy possessed. It is used in various compound terms such as pressure head, velocity head and loss of head.

heavy soil: (Obsolete in scientific use). A soil with a high content of the fine separates, particularly clay, or one with a high drawbar pull and hence difficult to cultivate. See fine texture.

heterotroph (organotroph); heterotrophic organism: An organism that obtains energy from organic compounds. An organism that utilizes an organic compound as its main source of carbon.

heterotrophic: Capable of deriving energy for life processes only from the decomposition of organic compounds and incapable of using inorganic compounds as sole sources of energy or for organic synthesis. Contrast with autotrophic.

horizon: See soil horizon.

host: An organism on or in which smaller organisms or viruses live, feed, or reproduce. When dealing with parasites having complex life cycles, the host in which the adult lives, or the one in which sexual reproduction takes place, is called the definitive host. A host harboring larval or asexually reproducing forms is called an intermediate host.

hydration: The physical binding of water molecules to ions, molecules, particles, or other matter.

hydraulic conductivity: See conductivity, hydraulic.

hydraulic gradient: The slope of the hydraulic grade line; the rate of change of pressure head; the ratio of the loss in the sum of the pressure head and a position head to the flow distance. For open channels, it is the slope of the water surface and is frequently considered parallel to the invert. For closed conduits under pressure, it is the slope of the line joining the elevations to which water would rise in pipes freely vented and under atmospheric pressure. A positive slope is usually one which drops in the direction of flow.

hydrolysis: The chemical reaction of a compound with water, whereupon the anion from the compound combines with the hydrogen and the cation from the compound combines with the hydroxyl from the water to form an acid and a base.

illuviation: The process of deposition of colloidal soil material, removed from one horizon to another in the soil; usually from an upper to a lower horizon in the soil profile. See eluviation.

immunity: State of protection; for example, state of protection against the mumps virus. Natural or innate immunity--protection that results from the genetic make up of the host; for example, domestic animals have an innate immunity to mumps virus. Acquired immunity--immunity gained as a result of exposure to an agent; for example, immunity to mumps virus is usually acquired (actively) by response to infection with the virus, but it may also be acquired (passively) by the administration of specific antibodies formed by another host.

immobilization: The conversion of an element from the inorganic to the organic form in microbial or plant tissue, thus rendering the element not readily available to other organisms or plants.

impervious: Resistant to penetration by fluids or by roots.

infection: Invasion of tissues (including skin or mucous membranes) by microbes with or without the production of disease.

infiltration: The downward entry of water into the soil.

influent: Water, wastewater or other liquid flowing into a reservoir, basin or treatment plant or any unit thereof.

inorganic matter: Chemical substances of mineral origin, or more correctly, not of basically carbon structure.

intermittent filter: A natural or artificial bed of sand or other fine-grained material to the surface of which wastewater is applied intermittently in flooding doses and through which it passes; opportunity is given for filtration and the maintenance of an aerobic condition.

ion: A charged atom, molecule or radical, the migration of which affects the transport of electricity through an electrolyte or, to a certain extent, through a gas. An atom or molecule that has lost or gained one or more electrons. By such ionization it becomes electrically charged. An example is the alpha particle.

ion exchange: A chemical process involving reversible interchange of ions between a liquid and a solid but no radical change in structure of the solid.

ions: Atoms that are positively charged (cations) because of the loss of one or more electrons, or that are negatively charged (anions) because of a gain in electrons.

isomorphous substitution: The replacement of an ion considered normal to a mineral structure by another during the formation of a mineral.

landscape: All the natural features, such as fields, hills, forests, water, etc., which distinguish one part of the earth's surface from another part. Usually that portion of land or territory which the eye can comprehend in a single view, including all its natural characteristics.

leach: To cause water or other liquid to percolate through something.

leaching: The removal of materials in solution from the soil.

lift, air: A device for raising liquid by injecting air in and near the bottom of a riser pipe submerged in the liquid to be raised.

lipid: Any of a diverse group of organic substances which are relatively insoluble in water but soluble in alcohol, ether, chloroform, or other fat solvents.

liquefaction: Act or process of liquefying or of rendering or becoming liquid; reduction to a liquid state.

liquor: Water, wastewater, or any combination; commonly used to designate liquid phase when other phases are present.

loading: The time rate at which material is applied to a treatment device involving length, area, or volume or other design factor.

loess: Material transported and deposited by wind and consisting of predominantly silt-sized particles.

lysimeter: A device for measuring percolation and leaching losses from a column of soil under controlled conditions.

manifold: A pipe fitting with numerous branches to convey fluids between a large pipe and several smaller pipes or to permit choice of diverting flow from one of several sources or to one of several discharge points.

manometer: An instrument for measuring pressure. It usually consists of a U-shaped tube containing a liquid, the surface of which in one end of the tube moves proportionally with changes in pressure on the liquid in the other end. Also, a tube type of differential pressure gage.

mapping unit: A soil or combination of soils delineated on a map and,where possible, named to show the taxonomic unit or units included. Principally, mapping units on maps of soils depict soil types, phases, associations, or complexes.

matric potential: See potential, soil water.

mechanical aeration: The mixing, by mechanical means, of wastewater and activated sludge in the aeration tank of the activated sludge process to bring fresh surfaces of liquid into contact with the atmosphere.

medium texture: The texture exhibited by very fine sandy loams, loams, silt loams, and silts.

membrane filter: A filter made of plastic with a known pore diameter. It is used in bacteriological examination of water.

mesh: One of the openings or spaces in a screen. The value of the mesh is usually given as the number of openings per linear inch. This gives no recognition to the diameter of the wire and thus the mesh number does not always have a definite relation to the size of the hole.

microorganism: Minute organism, either plant or animal, invisible or barely visible to the naked eye.

milligrams per liter: A unit of the concentration of water or wastewater constituent. It is 0.001 g of the constituent in 1,000 ml of water. It has replaced the unit formerly used commonly, parts per million, to which it is approximately equivalent, in reporting the results of water and wastewater analysis.

mineral: Any of a class of substances occurring in nature, usually comprising inorganic substances, such as quartz and feldspar, of definite chemical composition and usually of definite crystal structure, but sometimes also including rocks formed by these substances as well as certain natural products of organic origin, such as asphalt and coal.

mineralization: The conversion of an element from an organic form to an inorganic state as a result of microbial decomposition.

mineralogy, soil: In practical use, the kinds and proportions of minerals present in a soil.

mineral soil: A soil consisting predominantly of, and having its properties determined by, mineral matter. Usually contains < 20% organic matter, but may contain an organic surface layer up to 30 cm thick.

mixed liquor: A mixture of activated sludge and organic matter undergoing activated sludge treatment in the aeration tank.

Mollisol: Soil order consisting of soils having a thick A horizon with more than one percent organic matter and a base-saturation percentage above 50. Normally, they are formed under grass vegetation. Distinguished from Vertisols in that they are not self-inverting.

monosaccharide: A sugar, a simple carbohydrate, generally having the formula $C_nH_{2n}O_n$ where n can vary from 3 to 8. The most common are 5 and 6.

montmorillonite: An aluminosilicate clay mineral with a 2:1 expanding structure; that is, with two silicon tetrahedral layers enclosing an aluminum octahedral layer. Considerable expansion may be caused by water moving between silica layers of contiguous units.

morphology: See soil morphology.

mottling: Spots or blotches of different color or shades of color interspersed with the dominant color.

negative pressure: A pressure less than the local atmospheric pressure at a given point.

nitrification: The biochemical oxidation of ammonium to nitrate.

nonsettleable solids: Wastewater matter that will stay in suspension for an extended period of time. Such period may be arbitrarily taken for testing purposes as one hour.

organic matter: Chemical substances of animal or vegetable origin, or more correctly, of basically carbon structure, comprising compounds consisting of hydrocarbons and their derivatives.

organic nitrogen: Nitrogen combined in organic molecules such as proteins, amino acids.

organic soil: A soil which contains a high percentage (> 15% or 20%) of organic matter throughout the solum.

osmotic potential: See potential, soil water.

osmotic pressure: In concept, the force per unit area required to equal the attractive (hydration) force for water exerted by ions dissolved in a solution.

oven-dry soil: Soil which has been dried at 105° C until it reaches an essentially constant weight.

oxidation: (1) The burning or other conversion of an element to an oxide; (2) An increase in positive valence of an element or ion caused by electron loss.

oxidation process: Any method of wastewater treatment for the oxidation of the putrescible organic matter. The usual methods are biological filtration and the activated sludge process.

oxidation-reduction potential: The potential required to transfer electrons from the oxidant to the reductant and used as a qualitative measure of the state of oxidation in wastewater treatment systems.

oxygen demand: The quantity of oxygen utilized in the biochemical oxidation of organic matter in a specified time, at a specified temperature and under specified conditions. See BOD.

Ozone: Oxygen in molecular form with three atoms of oxygen forming each molecule (O_3).

parasite: An organism that lives in or on another organism (the host) and gains benefit at the expense of the host.

parent material: The unconsolidated mineral or organic matter from which soils are developed.

particle density: The mass per unit volume of individual particles; usually expressed as grams per cubic centimeter.

particle size: The effective diameter of a particle usually measured by sedimentation or sieving.

particle-size distribution: The amounts of the various soil separates in a soil sample, usually expressed as weight percentages.

parts per million (ppm): Measure of proportion by weight; equivalent to a unit of solute per million unit weights of solution. Milligrams per liter expressing the concentration of a specified component in a dilute sewage.

pasteurization: The processes of heating food or other substances under controlled conditions of time and temperature (for example, 63°C for 30 min) to kill pathogens and reduce the numbers of other microbes.

pathogenic: Causing disease. "Pathogenic" is also used to designate microbes which commonly cause infectious diseases, as opposed to those which do so uncommonly or never.

ped: A unit of soil structure such as an aggregate, crumb, prism, block, or granule, formed by natural processes (in contrast with a clod, which is formed artificially).

pedon: The smallest volume (soil body) which displays the normal range of variation in properties of a soil. Where properties such as horizon thickness vary little along a lateral dimension, the pedon may occupy an area of a square yard or less. Where such a property varies substantially along a lateral dimension, a large pedon several square yards in area may be required to show the full range in variation.

percolation: The flow or trickling of a liquid downward through a contact or filtering medium. The liquid may or may not fill the pores of the medium.

percolation, soil water: The downward movement of excess water through soil.

permeability, soil: The ease with which gases, liquids, or plant roots penetrate or pass through soil.

pH: A measure of acidity and alkalinity, neutrality being at pH 7; pH under 7 indicates an acid solution and pH over 7 an alkaline solution; the nearer the pH to 7 the weaker the acid or alkali. The reciprocal of the logarithm of the hydrogen-ion concentration. The concentration is the weight of hydrogen ions, in grams, per liter of solution. Neutral water, for example, has a pH value of 7 and a hydrogen ion concentration of 10^{-7}.

pH, soil: The degree of acidity or alkalinity expressed by the negative logarithm of the hydrogen-ion activity of a soil.

phosphate: A salt or ester of phosphoric acid.

photooxidation: A chemical reaction occurring as a result of absorption of light energy in the presence of oxygen. It is sometimes responsible for the death of microbes.

photosynthesis: The sum total of the metabolic processes by which light energy is utilized to convert CO_2 and a reduced inorganic compound to cytoplasm. $6H_2X + 6CO_2 \rightarrow C\ H_{12}O_6 + 6X$.

plaque (plack): A clear area in a lawn or a monolayer of cells. Viral plaques are created by viral lysis of infected cells within the clear area.

plastic soil: A soil capable of being molded or deformed continuously and permanently, by relatively moderate pressure, into various shapes. See consistence.

platy structure: Soil aggregates that are developed predominately along the horizontal axes; laminated; flaky.

plow pan: A compacted layer beneath the plow layer produced by pressure exerted on the soil during plowing.

polypeptide: A chain of amino acids bonded together by peptide bonds. The length varies from several amino acids to the length coded for by one gene.

polysaccharide: Long chains, branched or unbranched, of monosaccharide subunits.

point source: Any discernible confined and discrete conveyance from which pollutants are or may be discharged.

potential, soil water: The potential energy of a unit quantity of water produced by the interaction of the water with such forces as capillary (matric), ion hydration (osmotic), and gravity, expressed relative to an arbitrarily selected reference potential. In practical application, potentials are used to predict the direction and rate of water flow through soils, or between the soil and some other system, such as plants or the outer atmosphere. Flow occurs spontaneously between points of different water potential, the direction of flow being toward the site of lower potential.

pore-size distribution: The volume of the various sizes of pores in a soil. Expressed as percentages of the bulk volume (soil plus pore space).

pore space: Space in the soil not occupied by solid particles.

porosity: The total volume of pore space; usually expressed as a percentage of the total soil volume.

prismatic structure: A soil structural type with a vertical axis much longer than the horizontal axes and a flat or indistinct upper surface. See columnar structure.

profile, soil: A vertical section of the soil through all its horizons and extending into the parent material.

protein: A macromolecule containing one or more polypeptide chains.

publicly owned treatment works: Any devices and systems used in the storage and treatment of minicipal sewage which are owned and operated by a public entity.

puddled soil: A soil in which structure has been mechanically destroyed, which allows the soil to run together when saturated with water. A soil that has been puddled occurs in a massive nonstructural state.

purification: The removal of objectionable matter from water by natural or artificial methods.

raw wastewater: Wastewater before it receives any treatment.

recirculation: In the wastewater field, the refiltration of all or a portion of the effluent in a trickling filter to maintain a uniform high rate through the filter. Return of a portion of the effluent to maintain minimum flow is sometimes called recycling.

reduce: The opposite of oxidize. The action of a substance to decrease the positive valence of an ion.

reduction: The decrease in positive valence, or increase in negative valence, caused by a gain in electrons by an ion or atom.

returned sludge: Settled activated sludge returned to mix with incoming raw or primary settled wastewater.

roughing filter: A wastewater filter of relatively coarse material operated at a high rate to afford preliminary treatment.

runoff: That portion of the precipitation of an area which is discharged from the area through stream channels.

sand: (1) A soil separate consisting of particles between 0.05 and 2.0 mm in diameter. (2) A soil textural class.

sand filter: A filter in which sand is used as a filtering medium.

saturate: (1) To fill all the voids between soil particles with a liquid. (2) To form the most concentrated solution possible under a given set of physical conditions in the presence of an excess of the solute. (3) To fill to capacity, as the adsorption complex with a cation species; e.g., H-saturated, etc.

saturation: A condition reached by a material, whether it be in solid, gaseous or liquid state, that holds another material within itself in a given state in an amount such that no more of such material can be held within it in the same state. The material is then said to be saturated or in a condition of saturation.

scum: The layer or film of extraneous or foreign matter that rises to the surface of a liquid and is formed there.

secondary settling tank: A tank through which effluent from some prior treatment process flows for the purpose of removing settleable solids. See sedimentation tank.

secondary wastewater treatment: The treatment of wastewater by biological methods after primary treatment by sedimentation.

sedimentation: The process of subsidence and deposition of suspended matter carried by water, wastewater, or other liquids, by gravity. It is usually accomplished by reducing the velocity of the liquid below the point at which it can transport the suspended material.

sedimentation tank: A basin or tank in which water or wastewater containing settleable solids is retained to remove by gravity a part of the suspended matter. Also called sedimentation basin, settling basin, settling tank.

selective medium: A medium that has components which restrict growth to organisms of a particular type.

separate: See soil separates.

settleable solids: That matter in wastewater which will not stay in suspension during a preselected settling period, such as one hour, but either settles to the bottom or floats to the top.

sewerage: A comprehensive term which includes facilities for collecting, pumping, treating and disposing of sewage; the sewer system and the sewage treatment works.

silt: (1) A soil separate consisting of particles between 0.05 and 0.002 mm in diameter. (2) A soil textural class.

single-grained state: A nonstructural state normally observed in soils containing a preponderance of large particles such as sand. Because of a lack of cohesion, the sand grains tend not to assemble in aggregate form.

siphon: A closed conduit a portion of which lies above the hydraulic grade line, resulting in a pressure less than atmospheric and requiring a vacuum within the conduit to start flow. A siphon utilizes atmospheric pressure to effect or increase the flow of water through the conduit.

slope: Deviation of a plane surface from the horizontal. Slope is conventionally expressed in degrees, which are units of vertical distance for each 100 units of horizontal distance.

slow sand filter: A filter for the purification of water in which water. without previous treatment is passed downward through a filtering medium consisting of a layer of sand or other suitable material, usually finer than for a rapid sand filter and for 24 to 40 inches in depth.

sludge blanket: Accumulation of sludge hydrodynamically suspended within an enclosed body of water or wastewater.

soil: (1) The unconsolidated mineral material on the immediate surface of the earth that serves as a natural medium for the growth of land plants. (2) The unconsolidated mineral matter on the surface of the earth that has been subjected to and influenced by genetic and environmental factors of parent material, climate (including moisture and temperature), macro- and microorganisms, and topography, all acting over a period of time and producing a product-soil-that differs from the material from which it is derived in many physical, chemical, biological, and morphological properties.

soil classification: The systematic arrangement of soils into groups or categories on the basis of their characteristics. Broad groupings are made on the basis of general characteristics and subdivisions on the basis of more detailed differences in specific properties. The three higher categories, which are broadly defined, are orders, suborders, and great groups. The lowest category is the soil series, with each series consisting of many individual occurrences or bodies of soil that are very similar in most respects.

soil genesis: The formation of soils; the creation of new characteristics by soil-development processes.

soil horizon: A layer of soil or soil material approximately parallel to the land surface and differing from adjacent genetically related layers in physical, chemical, and biological properties or characteristics such as color, structure, texture, consistence, pH, etc.

soil morphology: The physical constitution, particularly the structural properties, of a soil profile as exhibited by the kinds, thickness, and arrangement of the horizons in the profile, and by the texture, structure, consistence, and porosity of each horizon.

soil map: A map showing the distribution of soil types or other soil mapping units in relation to the prominent physical and cultural features of the earth's surface.

soil moisture: Water contained in the soil.

soil separates: Groups of mineral particles separated on the basis of a range in size. The principal separates are sand, silt, and clay.

soil series: The basic unit of soil classification and consisting of soils which are essentially alike in all major profile characteristics although the texture of the A horizon may vary somewhat. See soil type.

soil solution: The aqueous liquid phase of the soil and its solutes consisting of ions dissociated from the surfaces of the soil particles and of other soluble materials.

soil structure: The combination or arrangement of individual soil particles into definable aggregates, or peds, which are characterized and classified on the basis of size, shape, and degree of distinctness.

soil suction: A measure of the force of water retention in unsaturated soil. Soil suction is equal to a force per unit area that must be exceeded by an externally applied suction to initiate water flow from the soil. Soil suction is expressed in standard pressure terms.

soil survey: The systematic examination, description, classification, and mapping of soils in an area.

soil texture: The relative proportions of the various soil separates in a soil.

soil type: In mapping soils, a subdivision of a soil series based on differences in the texture of the A horizon.

soil water: A general term emphasizing the physical rather than the chemical properties and behavior of the soil solution.

solids: Material in the solid state.

total: The solids in water, sewage or other liquids; includes suspended and dissolved solids; all material remaining as residue after water has been evaporated.

dissolved: Solids which are present in solution.

suspended: Solids which are physically suspended in water, sewage or other liquids. The quantity of material deposited when a quantity of water, sewage, or other liquid is filtered through an asbestos mat in a Gootch crucible.

volatile: The quantity of solids in water, sewage or other liquid lost on ignition of total solids.

solids-retention time: The average residence time of suspended soils in a biological waste treatment system, equal to the total weight of suspended solids in the system divided by the total weight of suspended solids leaving the system per unit of time (usually per day).

solum (plural: sola): The upper and most weathered part of the soil profile; the A and B horizons.

solution feeder: A feeder for dispensing a chemical or other material in the liquid or dissolved state to water or wastewater at a rate controlled manually or automatically by the quantity of flow. The constant rate is usually volumetric.

Standards Methods: Methods or analysis of water, sewage, sludge and industrial wastes approved by a Joint Committee of the American Public Health Association, American Water Works Association and the Federation of Sewage and Industrial Wastes Association.

sterilization: Rendering an object or substance free of all viable microbes. Practically speaking, to sterilize an object is to make it extremely improbable that a single living organism or virus remains.

structure, soil: See soil structure.

subsoil: In general concept, that part of the soil below the depth of plowing.

substrate: The substance on which an enzyme acts to form the product.

suction: See soil suction.

surface discharge: The disposal of wastewater (before or after treatment) to the land surface or into a receiving water body.

surface soil: The uppermost part of the soil, ordinarily moved in tillage, or its equivalent in uncultivated soils and ranging in depth from 3-4 inches to 8-10 inches. Frequently designated as the plow layer or the Ap horizon.

tensiometer: A device for measuring the negative hydraulic pressure (or tension) of water in soil in situ; a porous, permeable ceramic cup connected through a tube to a manometer or vacuum gauge.

tension, soil water: The expression, in positive terms, of the negative hydraulic pressure of soil water.

texture: See soil texture.

textural class, soil: Soils grouped on the basis of a specified range in texture. In the United States 12 textural classes are recognized.

tight soil: A compact, relatively impervious and tenacious soil (or subsoil) which may or may not be plastic.

till: (1) Unstratified glacial drift deposited directly by the ice and consisting of clay, sand, gravel, and boulders intermingled in any proportion. (2) To plow and prepare for seeding; to seed or cultivate the soil.

titer: The concentration of a substance in solution; for example, the amount of a specific antibody in serum, usually measured as the highest dilution of serum which will give a positive test for that antibody. The titer is often expressed as the reciprocal of dilution; thus a serum which gives a positive test when diluted 1:256, but not at 1:512, is said to have a titer of 256.

trickling filter: A filter consisting of an artificial bed or coarse material, such as broken stone, clinkers, slate, slats, brush or plastic materials, over which wastewater is distributed or applied in drops, films or spray from troughs, drippers, moving distributors or fixed nozzles and through which it trickles to the underdrains, giving opportunity for the formation of zoogleal slimes which clarify and oxidize the wastewater.

topography: The physical features of a landscape, especially its relief and slope.

topsoil: (1) The layer of soil moved in cultivation. See surface soil. (2) The A horizon. (3) The A1 horizon. (4) Presumably fertile soil material used to topdress roadbanks, gardens, and lawns.

total solids: The sum of dissolved and undissolved constituents in water and wastewater, usually stated in milligrams per liter.

ultraviolet (UV) light: Electromagnetic radiation with a wavelength between 175 and 350 nm (shorter than visible light). Certain wavelengths, absorbed by nucleic acids result in mutation and death.

unsaturated flow: The movement of water in a soil which is not filled to capacity with water.

vapor pressure: (1) the pressure exerted by a vapor in a confined space. It is a function of the temperature. (2) The partial pressure of water vapor in the atmosphere. Also see humidity. (3) Partial pressure of any liquid.

vector: An animal, often an insect, which carries an infectious agent from one host to another.

vehicle: An inanimate carrier of an infectious agent from one host to another.

virus: An obligate intracellular parasite consisting of a bit of nucleic acid, surrounded by a protein coat and sometimes enclosed by an envelope. Different viruses are capable of infecting animals, plants, and bacteria.

viscosity: The cohesive force existing between particles of a fluid which causes the fluid to offer resistance to a relative sliding motion between particles.

water content: As applied to soils work: the amount of water held in a soil expressed on a weight or volume basis. Conventionally, water contents are expressed relative to the oven-dry weight or volume of soil.

water table: That level in saturated soil where the hydraulic pressure is zero.

water table, perched: The water table of a discontinuous saturated zone in a soil.

wastewater: The spent water of a community. From the standpoint of source, it may be a combination of the liquid and water-carried wastes from residences, commercial buildings, industrial plants and institutions, together with any groundwater, surface water and storm water that may be present. In recent years, the word wastewater has taken precidence over the word sewage.